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FREDERICK HOUSE LAKE DAM
STRESS MEASUREMENT PROGRAM - JULY 1989

Report No 90-213-K

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ABSTRACT

A series of 19 overcoring stress measurements were made in the Frederick House Lake Dam during July 1989. Zones of high stress were found to exist in the dam.

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TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1

2.0 PROGRAM OUTLINE 1

3.0 THE TEST METHOD 2

4.0 TEST RESULTS 4

5.0 CONCLUSIONS 6

6.0 RECOMMENDATIONS 6

APPENDIX 1 United States Bureau Of Mines Borehole Deformation Gauge Test Method 9

APPENDIX 2 Field Test Results 10

APPENDIX 3 Biaxial Test Results 11

APPENDIX 4 Test Results - Plan View 12

APPENDIX 5 Test Results - Cross Section View 13

LIST OF FIGURES

Page

Figure 1 Location of Frederick House Lake Dam 9

Figure 2 In Situ Stress Measurement Terminology 10

Figure 3 Frederick House Lake Dam Location Of Overcore Tests - Cross Section View July 1989 11

Figure 4 Frederick House Lake Dam Location Of Overcore Tests - Longitudinal View July 1989 12

LIST OF TABLES

Table 1 Frederick House Lake Dam Stress Test Results 13

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FREDERICK HOUSE LAKE DAM
STRESS MEASUREMENT PROGRAM - JULY 1989

1.0 INTRODUCTION

The Frederick House Lake Dam is a control dam located on the Frederick House River approximately 55 km north east of Timmins (see Figure 1). The general configuration of the dam is shown in Figure 2. The problem at the dam in the past was that the stop logs were getting jammed in their slots. It is believed that movement of the dam structure itself is responsible for this problem although the actual mechanism that acts as a driving force is not definitely known. In an attempt to identify and quantify the source of this distress in the dam, this stress measurement program and a companion laboratory study on the concrete in the dam were undertaken. This report will deal only with the findings of the stress study while a separate report on the concrete test results will be issued by the Concrete Section, Civil Research Department, when the laboratory testing is completed in late 1991.

2.0 PROGRAM OUTLINE

The stress measurement program that was originally envisioned for the Frederick House Lake Dam was for three boreholes located in the centre section of each of the two wing walls of the dam. The locations of these holes and the proposed test locations is shown in Figure 3. At the pre-project site meeting it became apparent that the original stress

program would have to be modified in order to accommodate the grouting program which was planned by the Geotechnical Section, Civil Engineering and Architecture Department, to grout the leak at the north end of the dam. Free access to the north end of the dam, where the grouting was carried out, had to be maintained and for this reason borehole 3 had to be moved and boreholes 1 and 2 could not be drilled. The actual location of stress measurement borehole is also shown in Figure 3. The location is not ideal but it was the only location which allowed testing in the massive portion of the dam and still allowed access for the grouting crew. The biggest drawbacks to the location were:

1. The proximity to the sluicegate introduces stress concentrations in the concrete due to the geometry of the structure which are superimposed on the stress magnitudes.

2. Testing was not carried out in areas where stress magnitudes could be more clearly related to the observed behaviour of the dam.

3.0 THE TEST METHOD

A three component USBM stress gauge that measures the deformation of a pilot hole in three directions in a plane perpendicular to the borehole axis while it is overcored by a larger core bit was used in this project. The strains measured and the Young's modulus value determined in a biaxial test are combined in a formula based on a plane-strain analysis. The stresses contained in this report are the major secondary principal stress, P, and the minor secondary principal stress, Q. Past laboratory testing has shown these results to be with 15 percent of applied loads. These stresses are termed secondary principal stresses as they represent only one plane in a three-dimensional body. Three non-parallel boreholes would be required to determine the complete stress field in a solid body.

This method has been found to be the most useful in a dam structure for the following reasons:

1) Unlike the epoxy bond gauges, this method does not depend on obtaining a clean dry borehole,

2) the gauge is accurate, robust, and reliable,

3) the time required for a test is relatively brief, compared with other test methods.

The formulae used in this analysis is:

$$P=\frac{E}{6d(1-\gamma^2)}U_1+U_2+U_3+\frac{1}{\sqrt{2}}[(U_1-U_2)^2+(U_2-U_3)^2+(U_3-U_1)^2]^{1/2}$$

$$Q=\frac{E}{6d(1-\gamma^2)}U_1+U_2+U_3-\frac{1}{\sqrt{2}}[(U_1-U_2)^2+(U_2-U_3)^2+(U_3-U_1)^2]^{1/2}$$

$$\theta=1/2\ \tan^{-1}\frac{\sqrt{3}(U_2-U_3)}{2U_1-U_2-U_3}$$

where P = major secondary principal stress
Q = minor secondary principal stress
θ = angle between U_1 and P turned in a counterclockwise direction
E = the Young's modulus of elasticity of the material in which the stress measurements are made
γ = Poisson's ratio of the material in which the stress measurements are made
d = the diameter of the hole

U_1, U_2, U_3 = are the strains that have been measured on the three separate gauge axes

U_1, U_2, U_3 are the strains that have been measured on the three separate gauge axes

The relationship between P, Q and e are shown in Figure 2, Appendix 1.

4.0 TEST RESULTS

The results of the stress tests are given in Table 1 and the test locations are given in Figures 3 and 4. Appendix 2 contains the field test deformation curves and Appendix 3 contains the biaxial test curves. The orientations of the secondary stresses with respect to the sluiceway geometry in plan view are contained in Appendix 4 while the stress orientations, in plan view, are shown on a longitudinal section of the dam in Appendix 5. In total 15 tests were carried out in concrete and 4 tests were carried out in the foundation rock. The tests in the foundation rock were to determine if there was a change in the stress magnitudes and directions from those that were measured in the concrete structure itself.

The P stresses in the concrete ranged from 0.34 MPa in compression to 33.14 MPa in compression. The Q stresses ranged from -3.67 MPa in tension to 8.41 MPa in compression. The values of Young's modulus of elasticity for the concrete ranges from 18,803 MPa to 48,320 MPa with the lower values being between the depths of 8.90 m, (29 ft 2 in) and 12.95 m (42 ft 6 in). The stress values in the rock range from 2.76 MPa in tension to 11.90 MPa in compression for the P stress. The Q stress varies from 5.30 MPa in tension to 5.78 MPa in compression. The value of Young's modulus of elasticity ranges from 75,701 MPa to 86,384 MPa.

The results from the tests in the upper portions of the dam (tests 1-7) indicate very low stresses especially tests 3 and 4 which were made in an area that was bounded by two fractures or construction joints, at least on the face of the dam as shown in Figure 3 and which may have created a stress relieved zone.

The lower portion of the dam shows a different situation as indicated by the results of tests 8 to 15. Tests 8 to 13 are fairly closely spaced between the depths of 11.87 m to 14.86 m (38 ft 11 in - 48 ft 9 in) which is unusual but was deemed necessary when the P stress results from test 9 (P= 7.04 MPa, Q= 0.61 MPa) were found to be significantly higher than those of test 8 (P= 1.86 MPa, Q= 0.44 MPa). It was decided to carry out

a third test to confirm these results. Test 10 results were similar to test 9 although slightly higher, 9.58 MPa as opposed to 7.04 MPa but the Q stresses were almost the same, 0.69 MPa and 0.61 MPa. The main difference between tests 9 and 10 was the direction of P which changed from 111° to 175°. The significant change in direction resulted in test 11 being taken at a depth of 13.31 m (43 ft 8 in) and gave results of P= 33.14 MPa, Q= -0.49 MPa and a P orientation of 051°. These results are disturbing as the measured P stress is of the same magnitude as the unconfined compressive strength of the concrete. The very small Q stress of -0.49 MPa may be a result of the presence of a fracture that has allowed preferential strain relief in one direction and/or the result of Poisson's effect in view of such a large P stress. When the overcore was recovered it was discovered that a fracture did exist 38.1 cm (15 in) below the test and was oriented as dip= 55° and dip direction 272°, assuming a dam axis orientation of 345°. Tests 12 and 13, which were carried out below test 11 in order to confirm the very high P stress, showed that P decreases with depth. The Q stress increases sharply for test 12 but then also decreases with depth. The reason for this behaviour is not known but may be related to the fact that test 11, as shown in Figure 4, was very close to the base of the spillway and may be affected by the geometry of the dam at this point. Tests 14 and 15 continue the trend of decreasing stress with distance from the spillway.

Tests 16 to 19 were taken in the foundation rock to determine the stress conditions in the rock. Tests 16 and 17 gave stress results which are probably effected to a certain extent by the loading of the dam and the foundation treatment during construction. The two deepest tests, 18 and 19 show the stress magnitudes further decrease to the point where in test 19 both P and Q become tensile. There are fairly fresh fractures in the rock overcore and this is most likely the cause of this tensile field. The reason for this fresh fracturing in the rock is not known at present.

The possibility of the field test data being in error must be addressed. The accuracy of the overcoring technique which is carried out in accordance with ASTM specification D4623(1) is thought to be within 15% of the actual stresses as established by work by Austin (2) and Hooker et al (3).

The effects of temperature, Alkali Aggregate Reactivity (AAR) or foundation rock/dam interaction can only be understood by instrumenting the dam and monitoring its behaviour. The temperature of the structure can be easily monitored by means of thermocouples or similar devices. The effects of AAR and foundation rock/dam interaction would have to be monitored with in-situ strain cells, tiltmeters, extensometers

and crack meters. The reading of this instrumentation could be made using an automated data logging and remote monitoring system in much the same way that the water level data at the site are transmitted.

Several concrete cores from the overcore borehole were given to the Concrete Technology Section, Civil Research Department, to determine the rate of concrete growth. No significant growth has been detected to date. A separate report on this study will be released upon completion of the testing which should be by late 1991.

5.0 CONCLUSIONS

Based on our field and laboratory results and using classical stress analysis techniques and in answer to the concerns that this study had to address, it can be concluded that:

1. The alkali-aggregate reaction detected in the core samples taken from this hole is not extensive and, to date, laboratory testing does not indicate a high rate of concrete growth.

2. There are zones of very high stress in the dam and these could conceivably cause concrete creep

3. The stresses in portions of the dam are high enough to cause arching.

6.0 RECOMMENDATIONS

Keeping in mind the possibility that the dam may be converted at some future date to accept generating units, the following recommendations are made:

1. Install thermocouples, tiltmeters and in situ stress gauges to monitor the performance to the dam. These instruments would have to be remotely monitored in much the same way as water levels are in order to reduce data collection costs.

2. Consider stress testing in the locations as originally suggested. These tests could be carried out using standard overcoring techniques and then either install long-

term stress gauges or carry out repeat measurements in the same boreholes using a new gauge which has become available.

3. Geological mapping of the area around the dam to determine whether there are any indicators of geological structures which may be related to the behaviour of the dam.

The information obtained from the recommended actions would provide a basis for management decisions regarding remedial measures such as slot cutting.

The above recommendations are made prior to completion of the concrete study laboratory tests and may have to be modified as the long-term behaviour of the concrete becomes apparent.

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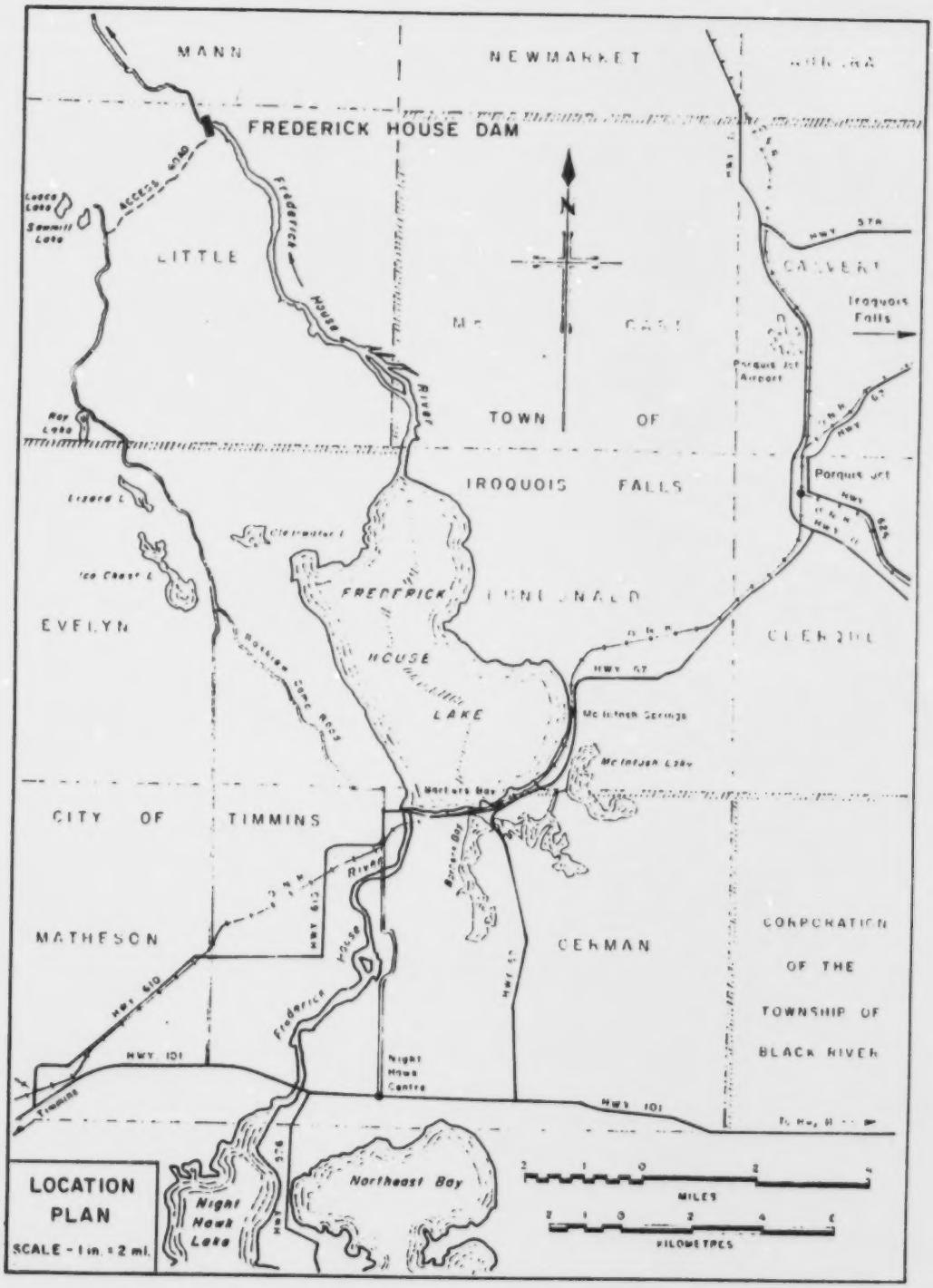

FIGURE 1

LOCATION OF FREDERICK HOUSE LAKE DAM

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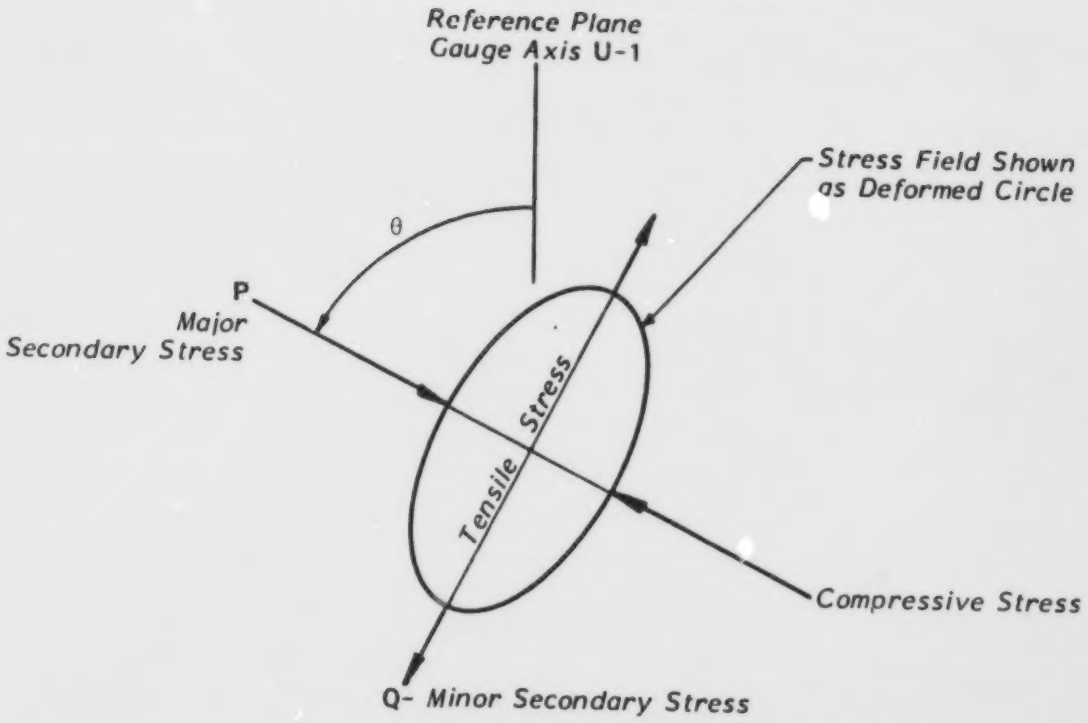

FIGURE 2
IN SITU STRESS MEASUREMENT TERMINOLOGY

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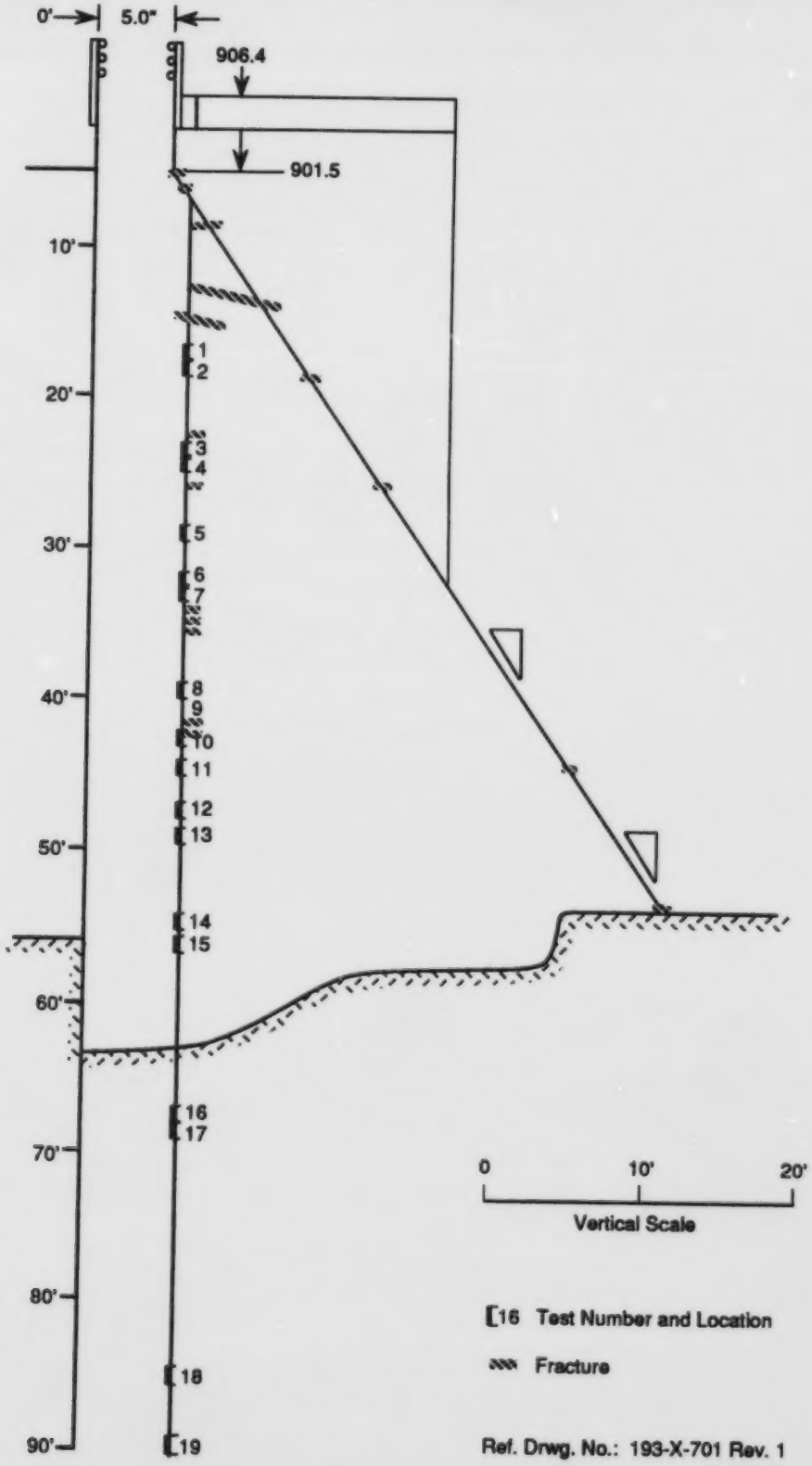

FIGURE 3

FREDERICK HOUSE LAKE DAM
LOCATION OF OVERCORE TESTS - CROSS SECTION VIEW
JULY 1989

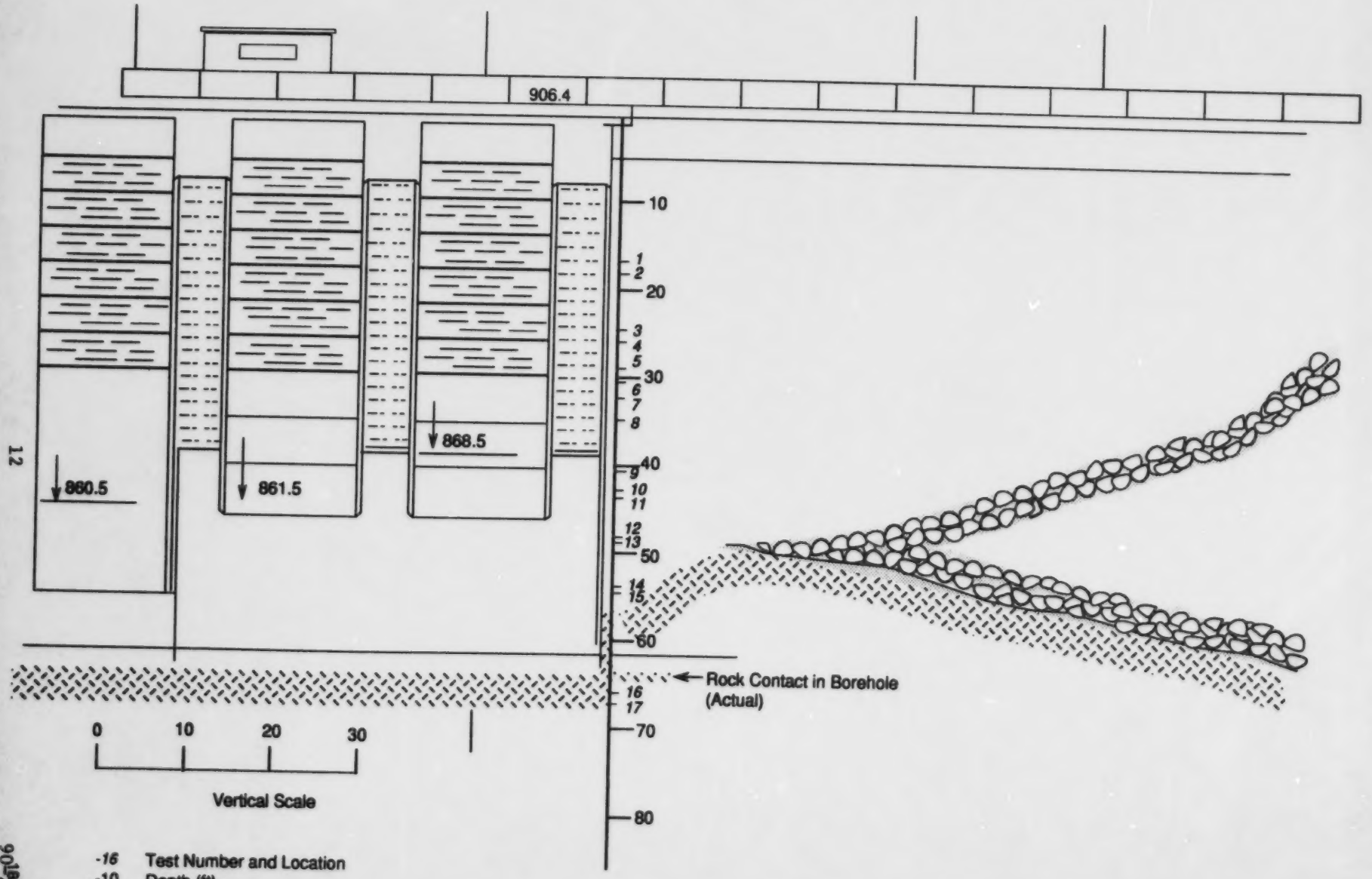

FIGURE 4

FREDERICK HOUSE LAKE DAM
LOCATION OF OVERCORE TESTS - LONGITUDINAL VIEW
JULY 1989

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TABLE 1

FREDERICK HOUSE LAKE DAM STRESS TEST RESULTS

TEST	DEPTH ELEVATION				P		Q		P/Q	AVG. E		AZIMUTH OF P
	m	Ft	m	Ft	MPa	psi	MPa	psi		MPa	X10⁶psi	
1	5.16	16' 11"	271.11	889' 6"	3.14	455	2.20	319	1.42	48320	7.01	123
2	5.67	18' 7"	270.60	887' 10"	1.95	283	0.57	83	3.41	45960	6.67	21
3	7.04	23' 1"	269.20	883' 4"	0.34	49	-0.07	-10	-4.69	32633	4.73	140
4	7.34	24' 1"	268.92	882' 4"	0.35	51	-3.67	-532	-0.10	32633	4.73	150
5	8.90	29' 2"	267.37	877' 3"	2.70	392	0.87	125	3.12	20187	2.93	88
6	9.64	31' 7"	266.64	874' 10"	0.37	54	0.11	16	3.25	18803	2.73	25
7	10.00	32' 10"	266.26	873' 7"	0.87	127	0.52	75	1.68	18803	2.73	169
8	11.87	38' 11"	264.40	867' 6"	1.86	270	0.44	63	4.25	25198	3.65	140
9	12.23	40' 1"	264.04	866' 4"	7.04	1021	0.61	88	11.61	25198	3.65	111
10	12.95	42' 6"	263.31	863' 11"	9.58	1389	0.69	100	13.95	22927	3.33	175
11	13.31	43' 8"	262.95	862' 9"	33.14	4806	-0.49	-71	-68.09	30395	4.41	51
12	14.40	47' 3"	261.86	859' 2"	11.97	1736	8.41	1220	1.42	33732	4.89	139
13	14.86	48' 9"	266.89	857' 8"	6.63	962	0.36	52	18.33	39899	5.79	159
14	16.78	55' 0"	259.50	851' 5"	4.21	611	-1.36	-198	-3.09	30747	4.46	151
15	17.22	56' 6"	259.04	849' 11"	2.26	328	0.52	75	4.39	24570	3.56	135
16	20.15	66' 1"	256.12	840' 4"	9.53	1382	-2.27	-330	-4.19	85031	12.33	133
17	20.69	67' 10"	255.59	838' 7"	11.90	1726	5.78	838	2.06	86384	12.53	121
18	25.92	85' 0"	250.36	821' 5"	0.34	50	-5.30	-768	-0.07	80830	11.72	124
19	26.92	88' 4"	249.34	818' 1"	-2.76	-401	-4.79	-695	0.58	75701	10.98	137

APPENDIX 1

UNITED STATES BUREAU OF MINES BOREHOLE DEFORMATION GAUGE TEST METHOD

THE UNITED STATES BUREAU OF MINES BOREHOLE DEFORMATION GAUGE TEST METHOD

The United States Bureau of Mines Borehole Deformation Gauge (USBM gauge) was chosen to measure the stresses existing in rock masses and concrete structures because past experience has shown it to be a reliable instrument. The method consists of drilling an HQ hole (diameter 97.5 mm), drilling a 10 cm long E pilot hole (diameter 38.1 mm), extending the E hole to approximately 60 cm in length, placing the USBM gauge into the hole and then overcoring the gauge with a thin wall bit. Strain readings are taken during this overcoring for each 13 mm of bit advance.

The accuracy for this test method claimed by the US Bureau of Mines is that the stresses measured are within 15% of actual stresses in concrete and slightly better in rock.

TEST THEORY

Strain relief measurements are made along three separate axes which form a 60° strain rosette. After the downhole testing is completed, the overcore is recovered and subjected to testing in a biaxial chamber to determine the Young's modulus of the material. This result is combined with the downhole strain relief measurements to calculate the secondary principal stresses existing in the plane perpendicular to the borehole axis. The maximum secondary principal stress (P) and the minimum secondary principal stress (Q) are termed secondary because they represent the maximum and minimum stress in one plane within a three dimensional rock mass. The relationship of P,Q and θ is shown in Figure 1. Three non-parallel boreholes would be required to completely define the three dimensional stress field using this test method.

The analytical method used to calculate the stresses is based on elastic thick wall cylinder theory using a 60° strain rosette solution. The equations used are:

$$P = \frac{E}{(1-\nu^2)d}\left[U_1+U_2+U_3+\frac{1}{\sqrt{2}}\left[(U_1-U_2)^2+(U_2-U_3)^2+(U_3-U_1)^2\right]^{1/2}\right]$$

$$Q = \frac{E}{(1-\nu^2)d}\left[U_1+U_2+U_3-\frac{1}{\sqrt{2}}\left[(U_1-U_2)^2+(U_2-U_3)^2+(U_3-U_1)^2\right]^{1/2}\right]$$

$$\theta = 1/2\ \tan^{-1}\frac{\sqrt{3}\ (U_2-U_3)}{2U_1-U_2-U_3}$$

where P is the major secondary principal stress
Q is the minor secondary principal stress
θ is the angle measured from gauge axis U_1 to P in a counter-clockwise direction
E is the Young's Modulus of elasticity of the rock in which the measurement was made (Note: refer to Biaxial Testing Section for a description of this test method)
ν is the Poisson's ratio of the rock
d is the diameter of the borehole in which the gauge was placed
U_1, U_2, U_3 - are the strains measured along the three gauge axes and are positive for an increasing diameter

Deformation Curves

Prior to the computation of the stresses, the test data is plotted to ensure that no unusual downhole behaviour such as drill vibration is misinterpreted as actual strain relief. An example of these curves which plot deformation versus the depth of bit is given in Figure 2.

GAUGE PLACEMENT AND ORIENTATION

The placement of the gauge is accomplished by means of a series of keyed aluminum rods that attach to a placing tool (Figure 3). The gauge is fastened to the placing tool and lowered into the HQ hole until just above the E hole. Prior to the placement of the gauge the wall of the E pilot is scribed using the scribing tool shown in Figure 4. The hardened steel point is aligned with a scope mounted on top of the rods before it is pushed into the hole as shown in Figure 4. The mark enables proper orientation of the gauge in the core during the biaxial test to determine the Young's Modulus of Elasticity of the core in the same directions as the downhole strain readings were made. The scope, as shown in Figure 5, is used to sight a predetermined reference point so that the azimuth of P can be calculated. The gauge is aligned in the same manner.

Drilling Procedure

The drilling procedure for this test makes use of a modified wire line system that was developed by Longyear Canada during several projects at or near the Darlington NGS site. A crawler mounted CME drill equipped with a hydraulic chuck is used as this requires a drill set up time of less than one half day. The hydraulic chuck enables the rods to be easily centered in the hole each time. The drill set up is shown in Figure 6.

The procedure used in preparing the hole for an overcore test is outlined below. The sequence of operations is shown in Figure 7.

1) The hole is advanced to a depth that is 20cm above the predetermined test depth by normal wire line drilling practice using an HQ bit.

2) The HQ standard HQ inner barrel assembly is removed and replaced by a modified assembly that is shown in Figures 8-11. The barrel does not rotate and enables this barrel to be used to drill the E hole.

3) A 10 cm (4 in) E starter barrel is attached to the inner tube and lowered into the hole. A 10 cm starter hole is drilled after the H rods are pulled back approximately 15 cm to ensure the core barrel dogs are engaged.

4) The inner barrel assembly is removed and the starter barrel is replaced by a 60 cm E core barrel. After encountering difficulty in obtaining E core that indicated natural discontinuities and breaks in the rock as opposed to breaks that are caused by drilling, it was decided to switch from a single barrel E core to an IEWS barrel that is shown in Figures 10 and 11. The IEWS barrel has an inner tube that is free to swivel and greatly reduces core breakage due to vibration during drilling and enabled a truer estimation of rock conditions to be made. This improvement enables the avoidance of zones of poor rock which would have jeopardized the gauge with a core breakage during the test. In this step, as in step 3, the H rods are raised to engage the dogs on the core barrel to enable drilling of the E hole.

5) The E core is carefully inspected to ensure that it is possible to carry out a test. It is necessary to have at least 20 cm of solid rock below the buttons. These lengths of rock are necessary to ensure that the complete stress relief action of the core can be monitored during the test.

6) The HQ rods are pulled and the HQ bit is replaced by the thin wall industrial core bit.

7) The E hole is scribed with the scriber shown in Figure 4.

8) The gauge is lowered through the water swivel and drive rod, then attached to the placement tool, lowered down the hole, oriented and then pushed into the E hole.

9) The end plug, through which the electrical readout cable is run, is screwed into the water swivel. The readout cable is then connected to the Vishay strainmeters.

10) The gauge is allowed to thermally stabilize in the E hole for five minutes.

11) The drill water pressure and drill speed are adjusted.

12) The overcoring begins with strain readings taken for every 1.27 cm (0.5 in) of bit advance.

13) At the completion of the overcoring the gauge is removed.

14) The H rods are pulled from the hole and the core breaker is lowered down the hole.

15) The core breaker is removed from the hole and the core catcher is lowered to retrieve the core.

16) The overcore is removed and prepared for biaxial testing.

17) The HQ bit and rods are lowered down the hole and the hole is extended down to the next test horizon.

Biaxial Testing

After completion of the down hole overcoring, the core is prepared for testing in the biaxial chamber. The chamber which is shown in Figure 12 consists of a hollow aluminum cylinder lined with a rubber membrane. A known hydraulic pressure is applied with a hand operated hydraulic pump. The pressure in the system is monitored by means of a gauge attached to the pump.

Once the core has been placed into the chamber and the gauge is inserted into the overcore using the scribe mark as an orientation guide, the system is bled of air bubbles. The core is cycled three times and the strains measured by the gauge monitored on the Vishay readouts. This enables the value of Young's modulus of the core to be determined.

The Young's Modulus of Elasticity (E) is calculated using the formula:

$$E = \frac{4ab^2P_0}{(b^2-a^2)U}$$

where a = inner radius of overcore
b = outer radius of overcore
P_0 = applied hydraulic pressure
U = diametral borehole deformation

When the overcore is tested, the E value is determined along each of the three axes and if it is found that the three values E_1, E_2 and E_3 vary by more than 25%, then it is necessary to apply an anisotropic solution to the field test data.

Only two anisotropic elastic constants of the material are necessary for the plane strain evaluation of secondary stresses (Hooker & Johnson, 1969). These parameters are:

$$\alpha_{11} = \frac{1 - \gamma_{13}\,\gamma_{31}}{E_1}$$

$$\alpha_{22} = \frac{1 - \gamma_{23}\,\gamma_{32}}{E_2}$$

where E_i = Young's modulus in the i direction
γ_{ij} = Poisson's ratio in the ij plane

The above subscripts refer to the orthogonal coordinate system defined by the axis of the core and the directions of maximum and minimum Young's modulus in the plane perpendicular to the axis of the core. This procedure avoids errors in the field data that are due to elastic non-linearity in the rock. This method involves many calculations and subsequent laboratory tests. Tullis (1981) found that, by using what he termed an "approximate" anisotropic solution, it was possible to obtain results that varied little from the Hooker and Johnson (1969) solution and yet avoided the cumbersome analysis and testing. This method was applied to all of the tests results to correct for the effect of rock anisotropy. The equation is:

where $U_i^m = U_i(E_i/E)$ $i = 1,2,3$
E_i = is Young's modulus for each axis
E = is the average Young's modulus for the three axes
U_i = is the deformation for measured on axis i
U_i^m = is the modified deformation on axis i

The modified deformations are then substituted into the original equations, (1), (2) and (3), for P,Q and θ and new values of P,Q and θ are calculated. The results of the biaxial tests are plotted as load versus deformation such as those shown in Figure 13, to ensure that the rock material is behaving elastically. Readings are taken during the loading and unloading portion of each of the three load-unload cycles. The unloading cycle E value is used in stress calculations because this simulates the downhole stress relief of the rock core as it is overcored.

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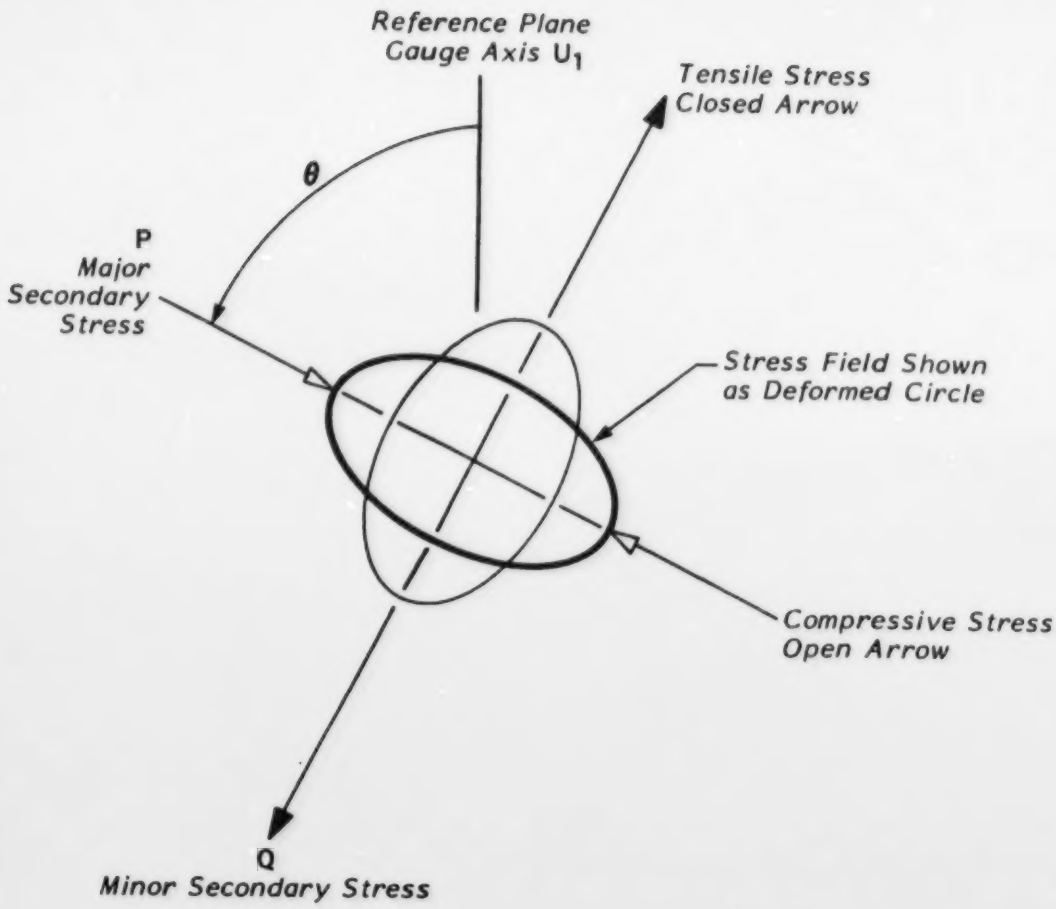


FIGURE 1

IN SITU STRESS TERMINOLOGY

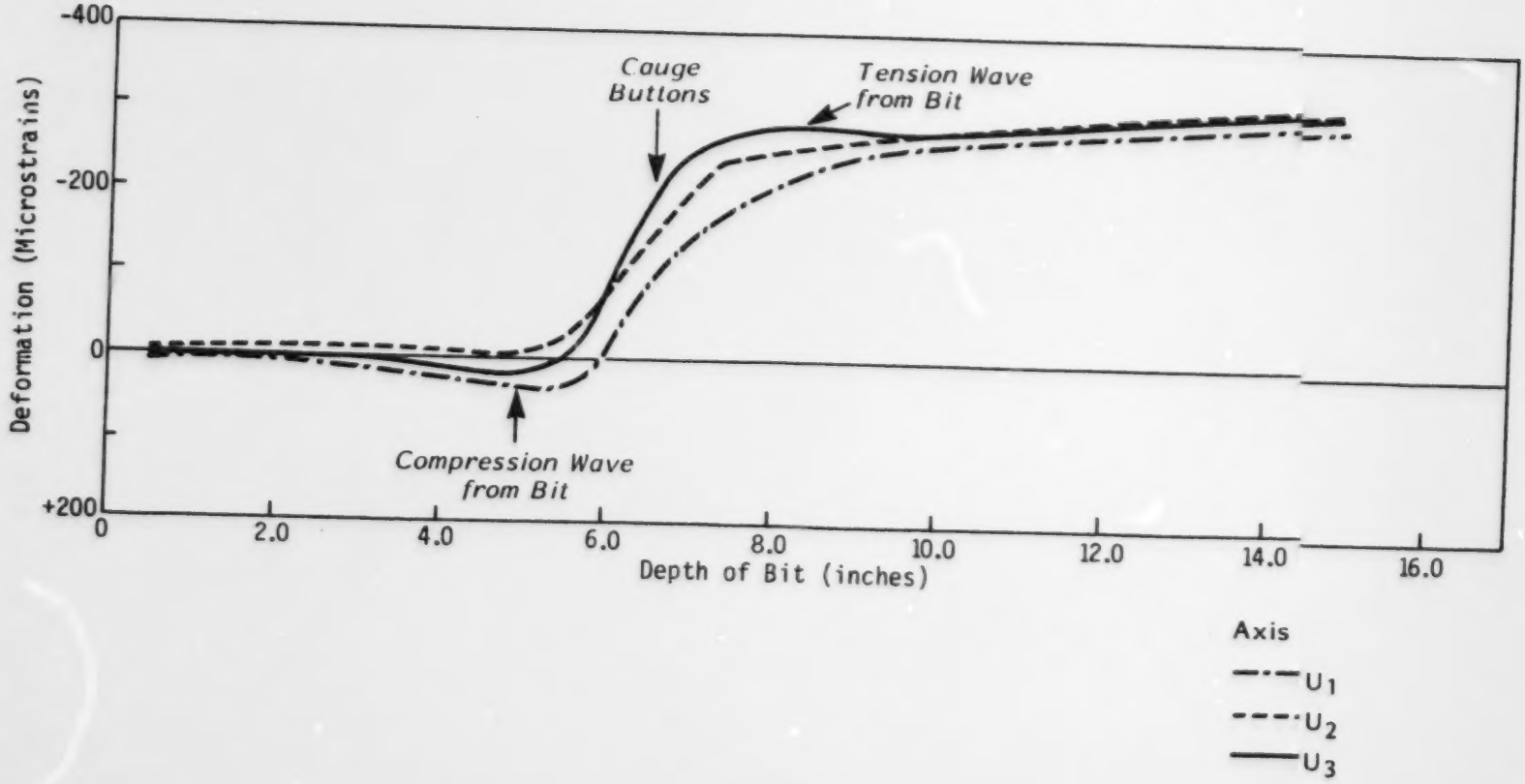
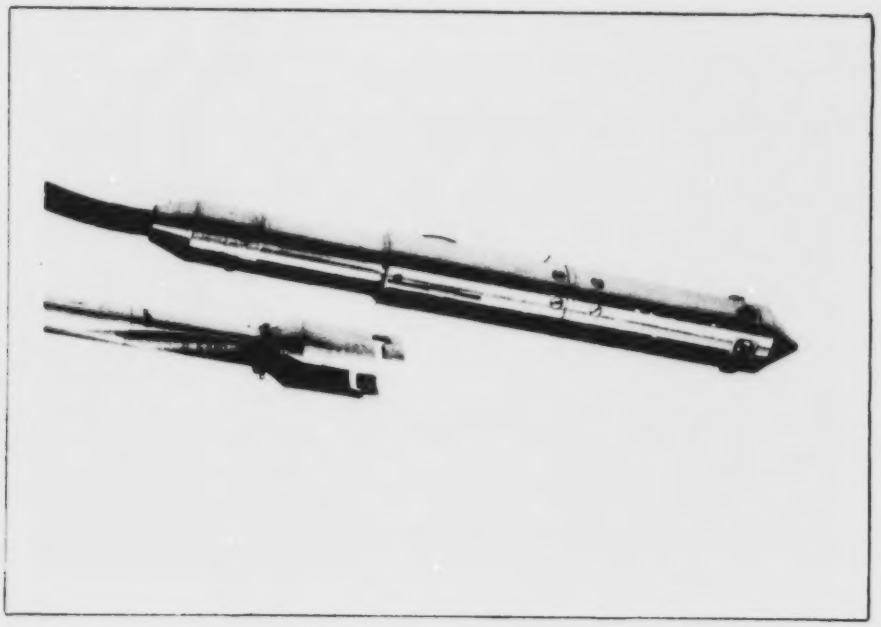


FIGURE 2
TYPICAL DEFORMATION CURVES FROM OVERCORING TEST



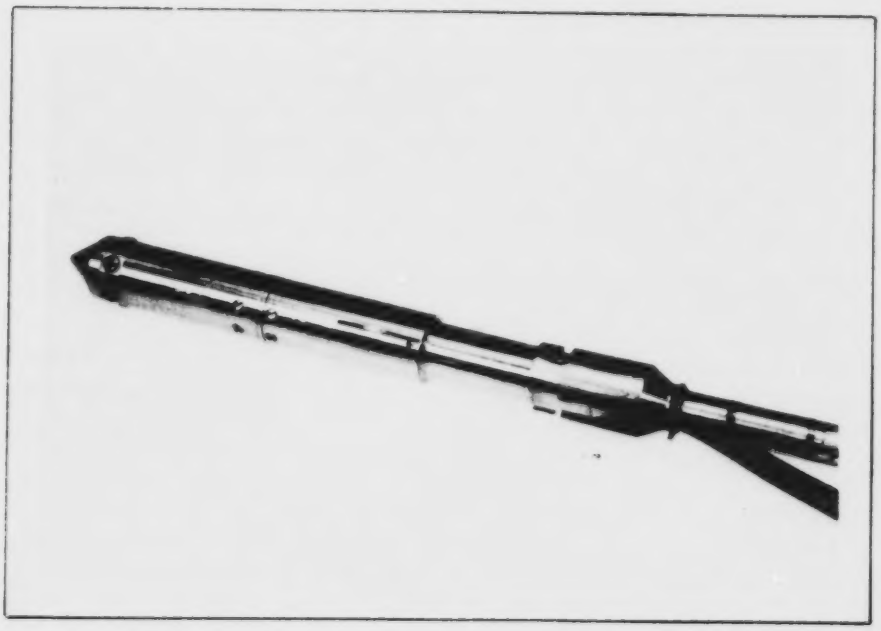


FIGURE 3
PLACING TOOL FOR USBM GAUGE

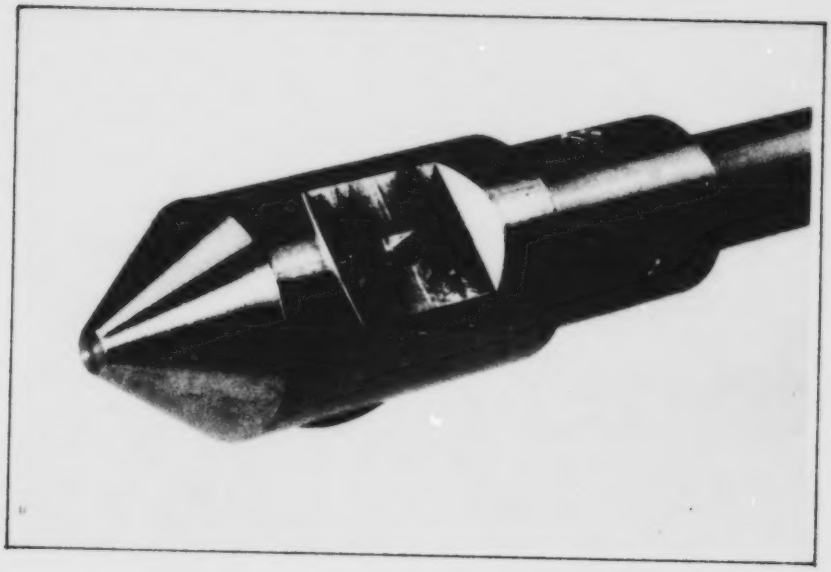


FIGURE 4
STEEL SCRIBING POINT

FIGURE 5
ORIENTATION OF THE GAUGE

FIGURE 6

DRILL RIG SET UP FOR OVERCORING

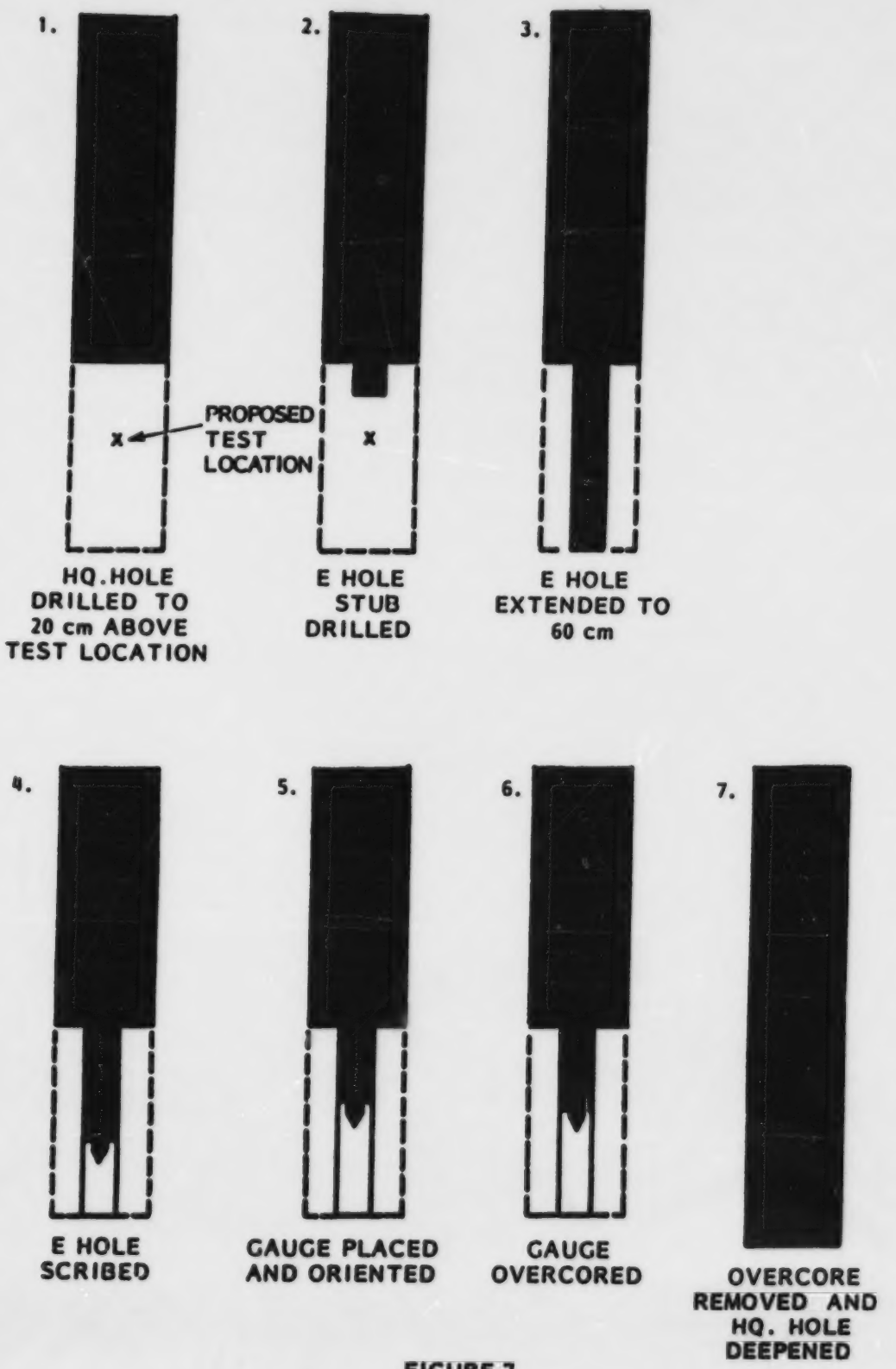


FIGURE 7

SEQUENCE FOR OVERCORE TEST USING HQ. WIRELINE SYSTEM

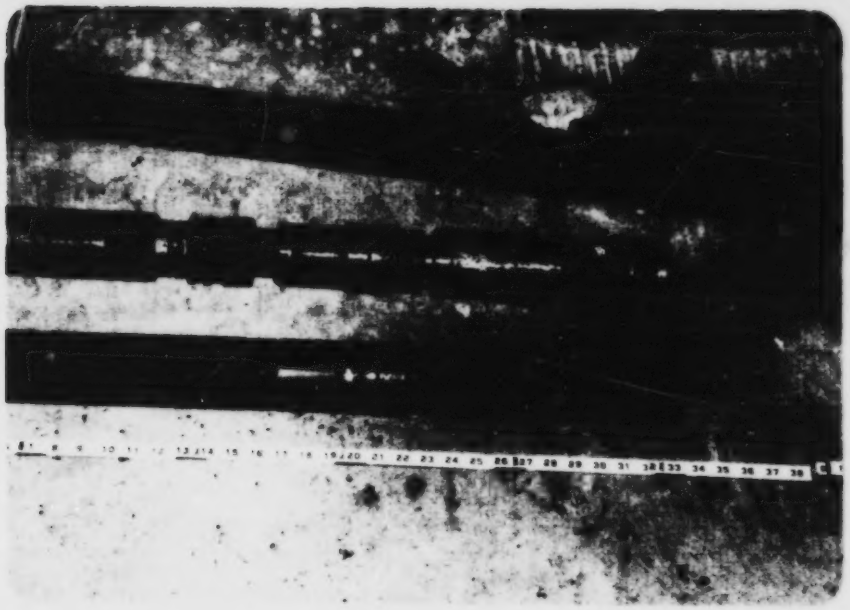


FIGURE 8

TOP STANDARD INNER BARREL ASSEMBLY
BOTTOM MODIFIED INNER BARREL ASSEMBLY

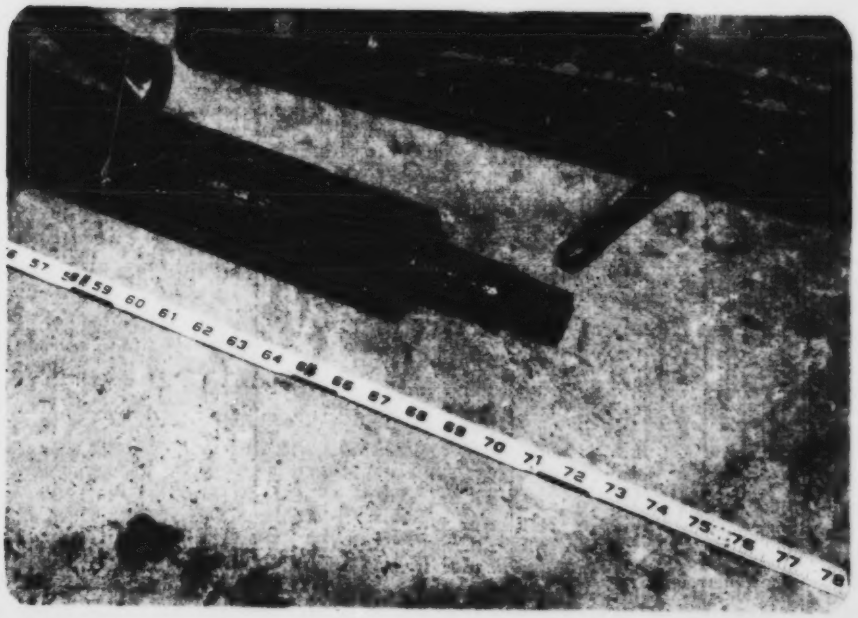


FIGURE 9

LOWER END OF MODIFIED INNER
BARREL ASSEMBLY WITH E BARREL
ADAPTOR IN PLACE

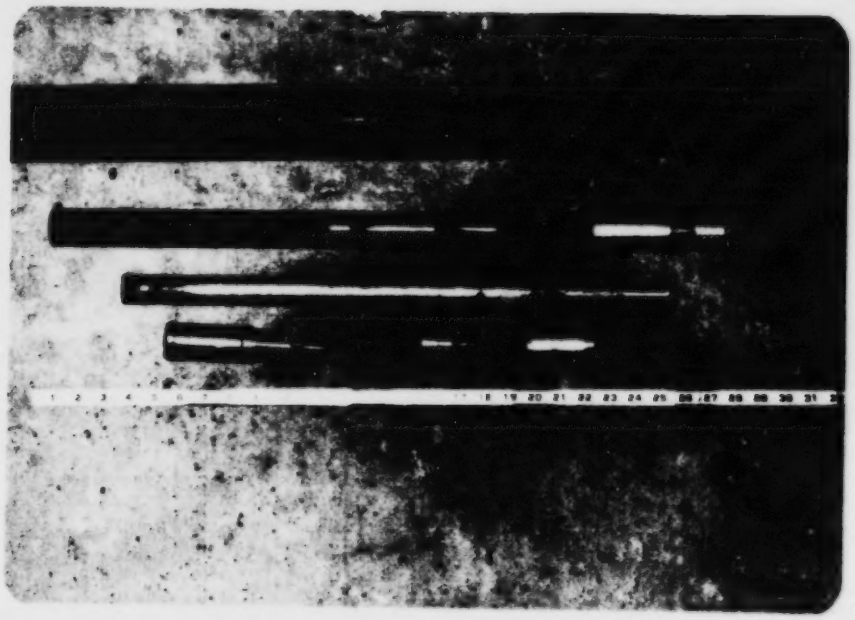


FIGURE 10

DISASSEMBLED IEWS CORE BARREL

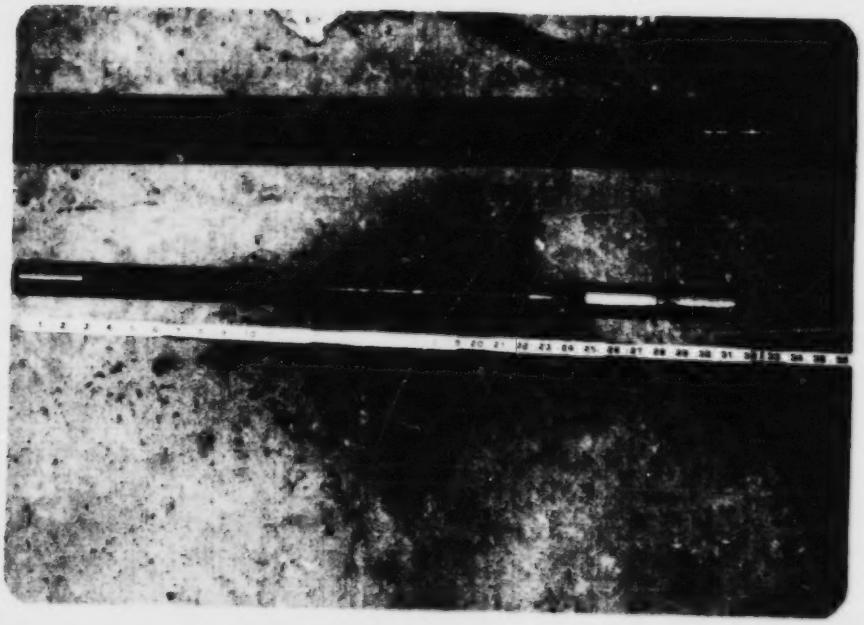


FIGURE 11

ASSEMBLED IEWS CORE BARREL

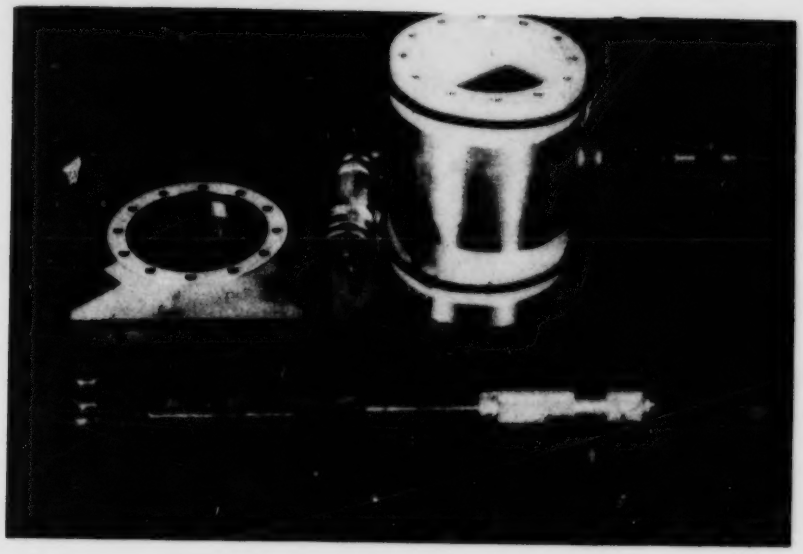


FIGURE 12
BIAXIAL CELL FOR YOUNG'S MODULUS DETERMINATION

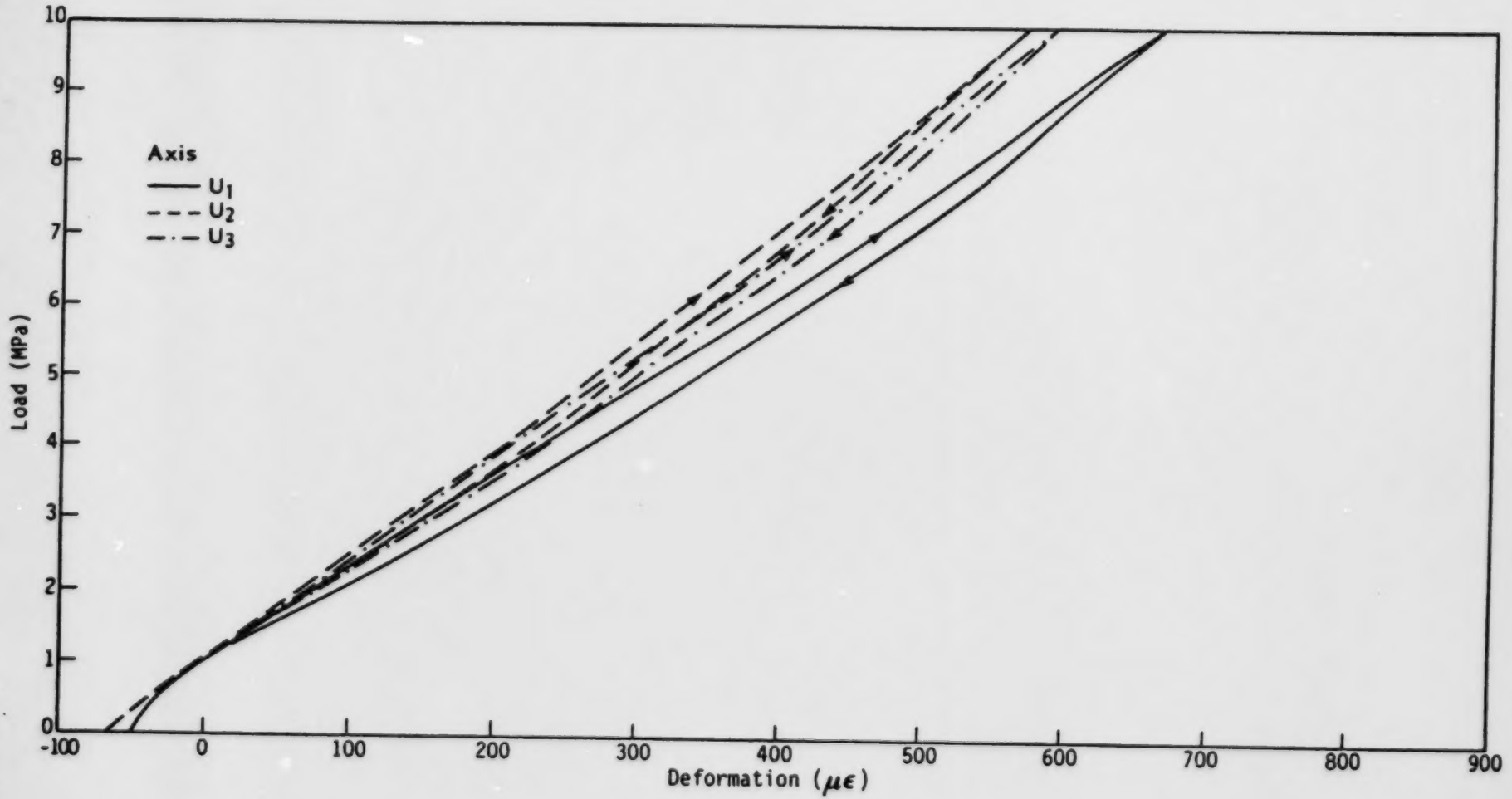


FIGURE 13

TYPICAL BIAXIAL TEST RESULTS

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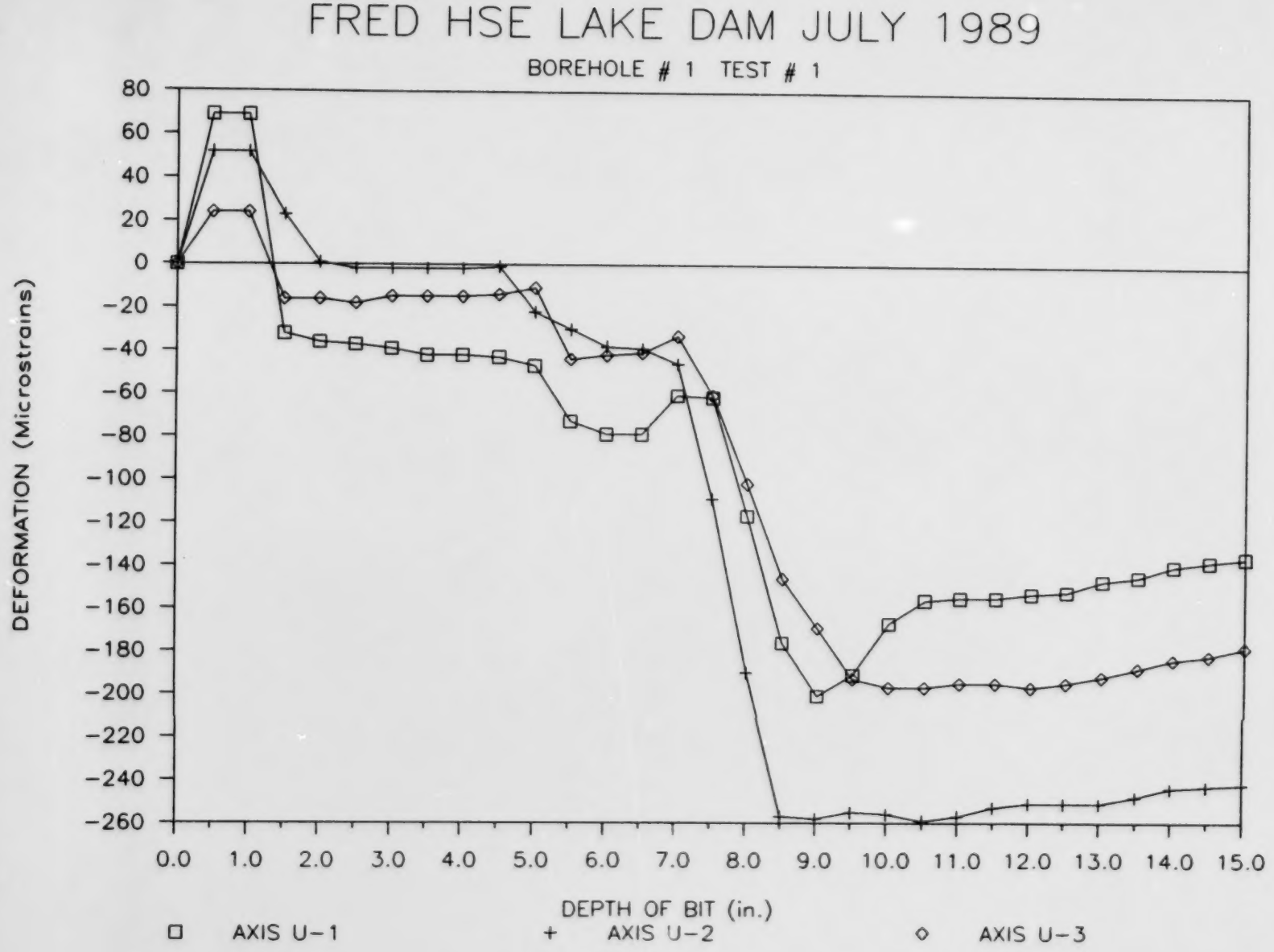
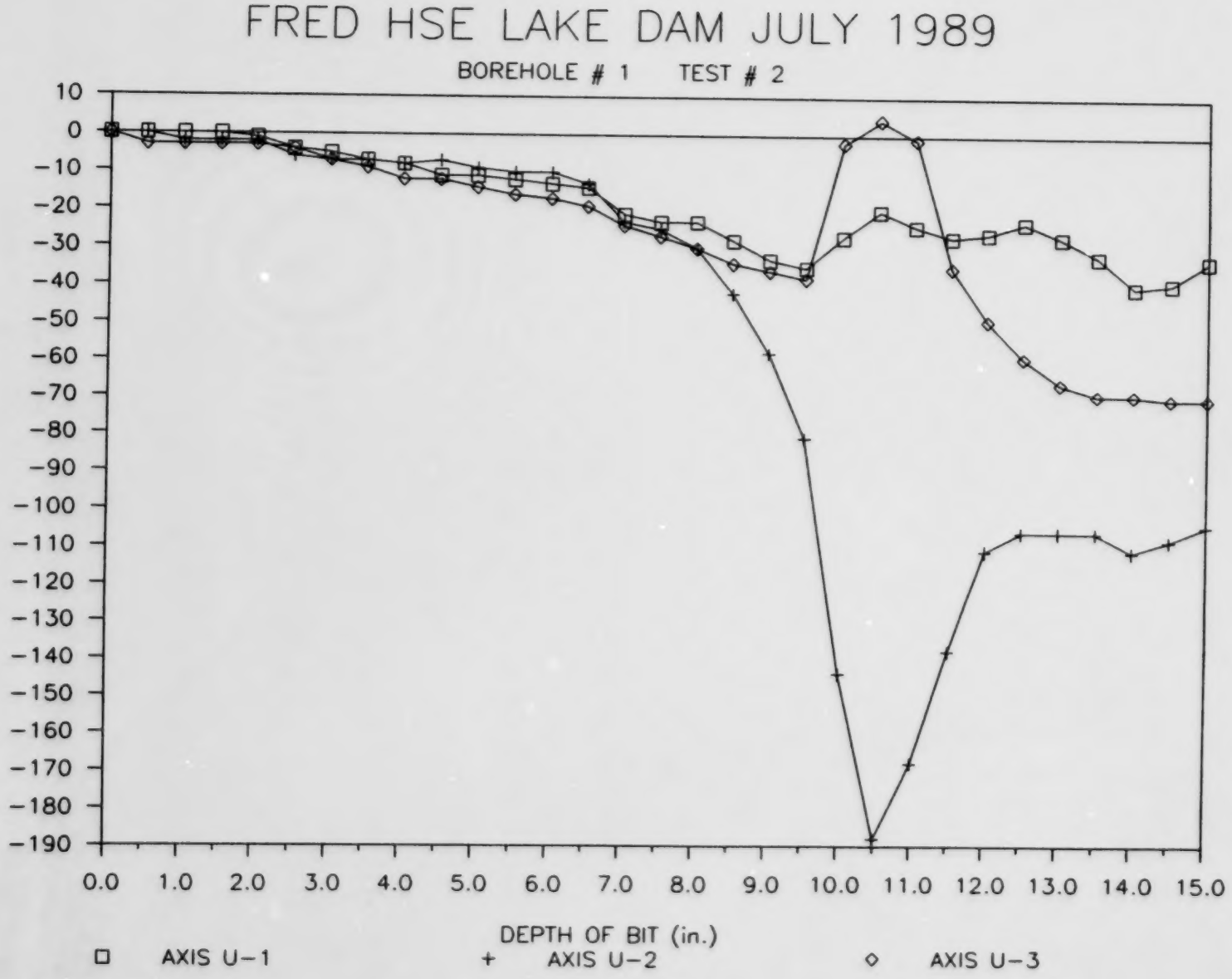
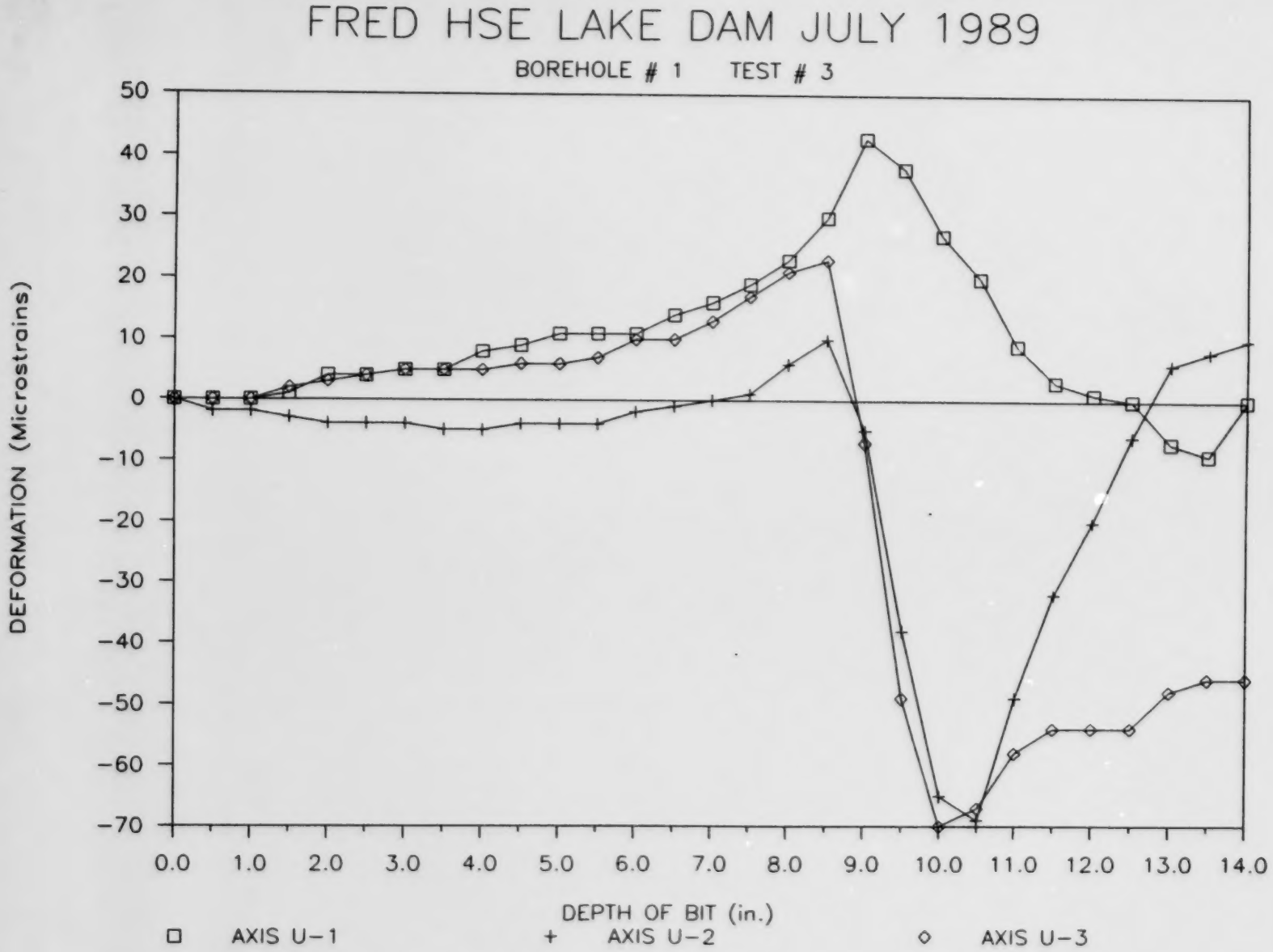
APPENDIX 2

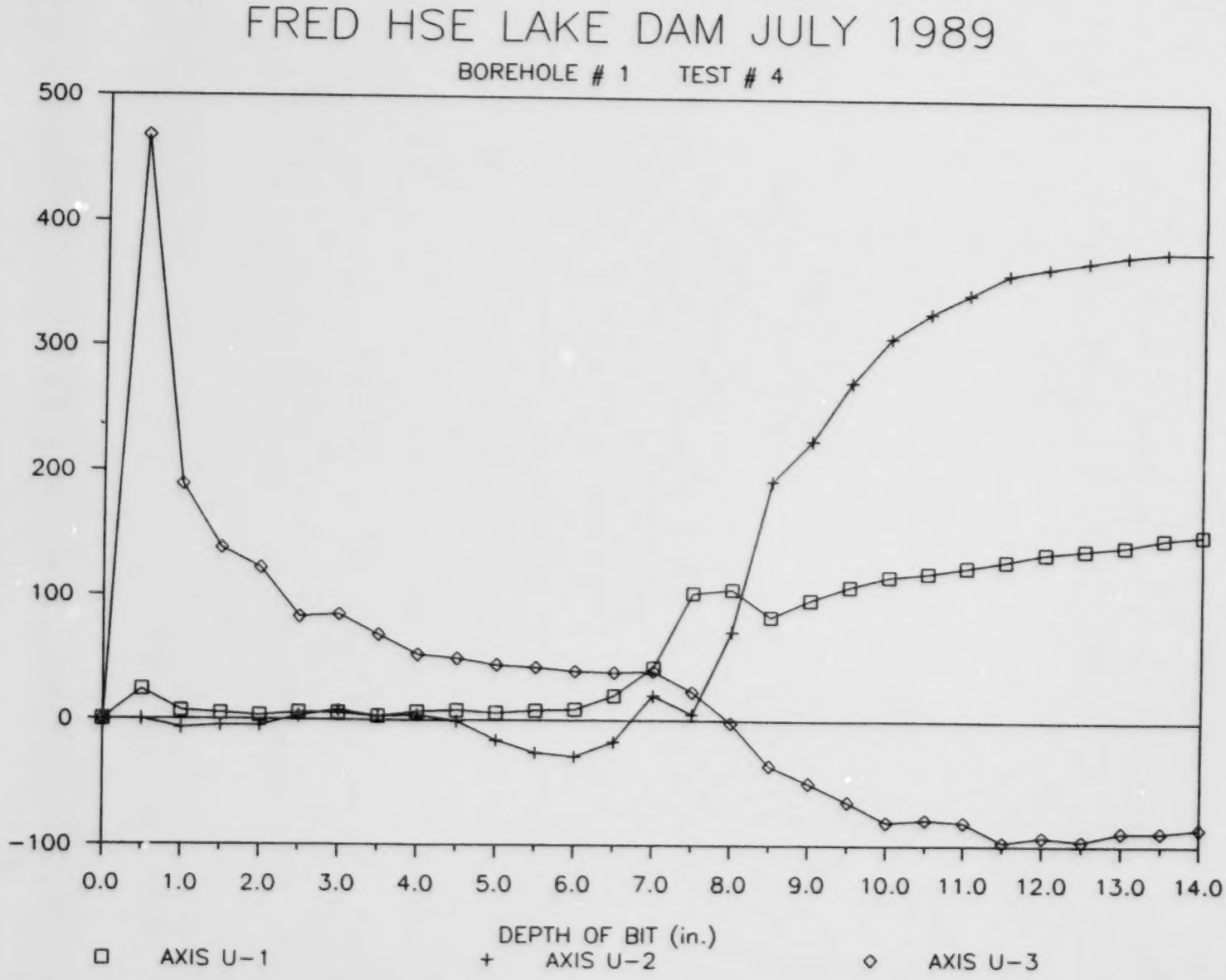
FIELD TEST RESULTS

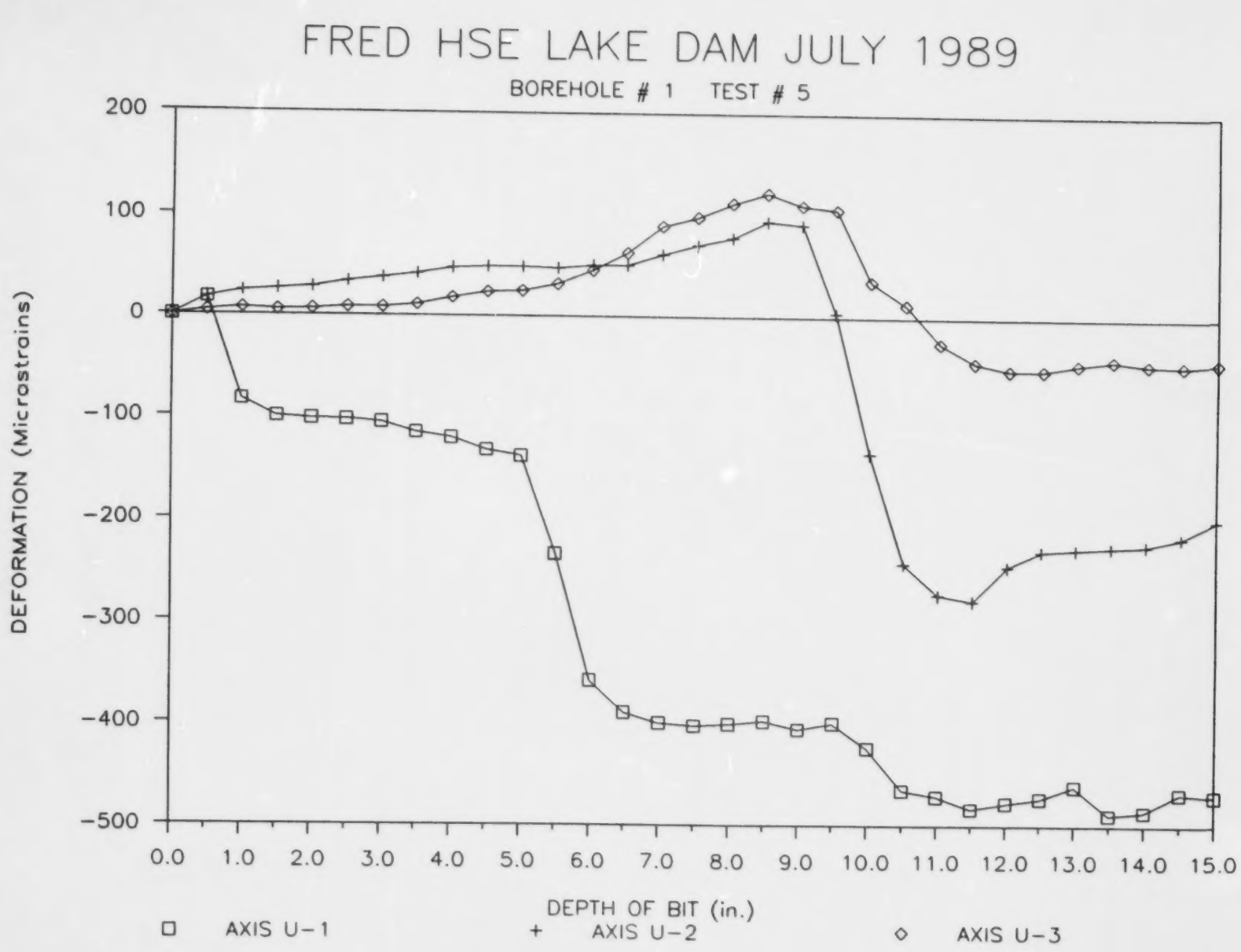

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 1
DEFORMATION (Microstrains)
80
60
40
20
0
-20
-40
-60
-80
-100
-120
-140
-160
-180
-200
-220
-240
-260
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

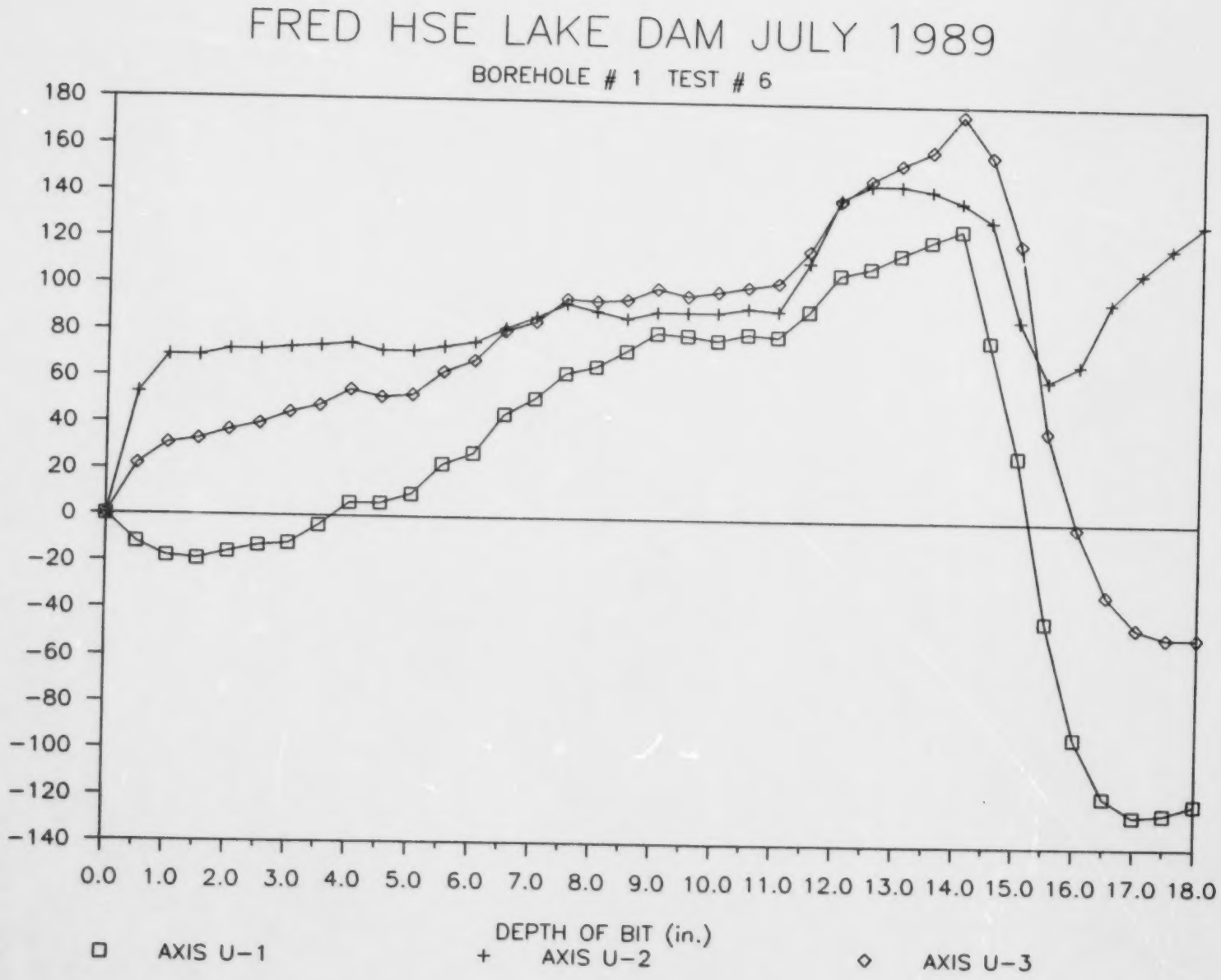

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 2
DEFORMATION (Microstrains)
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-110
-120
-130
-140
-150
-160
-170
-180
-190
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

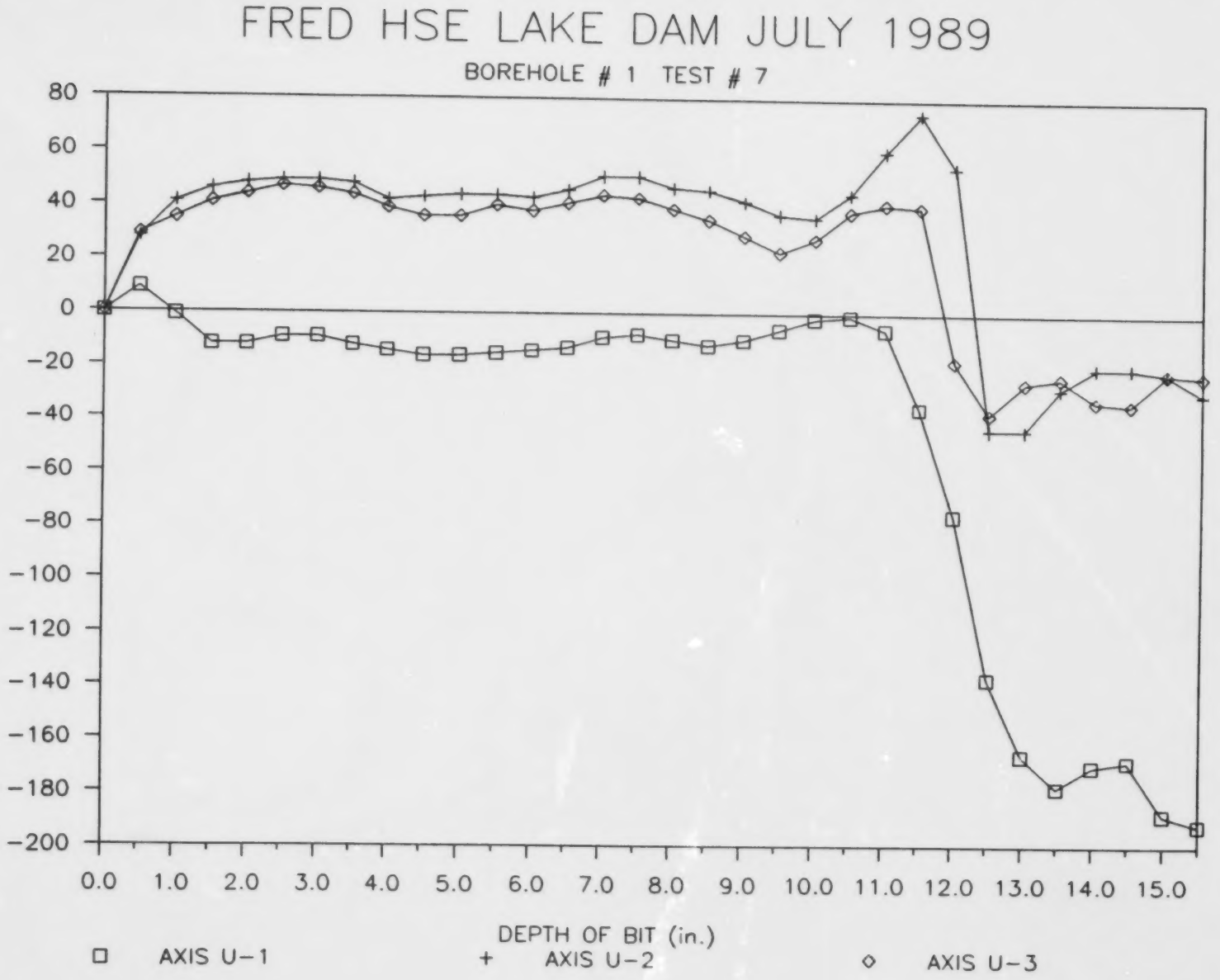
90-213

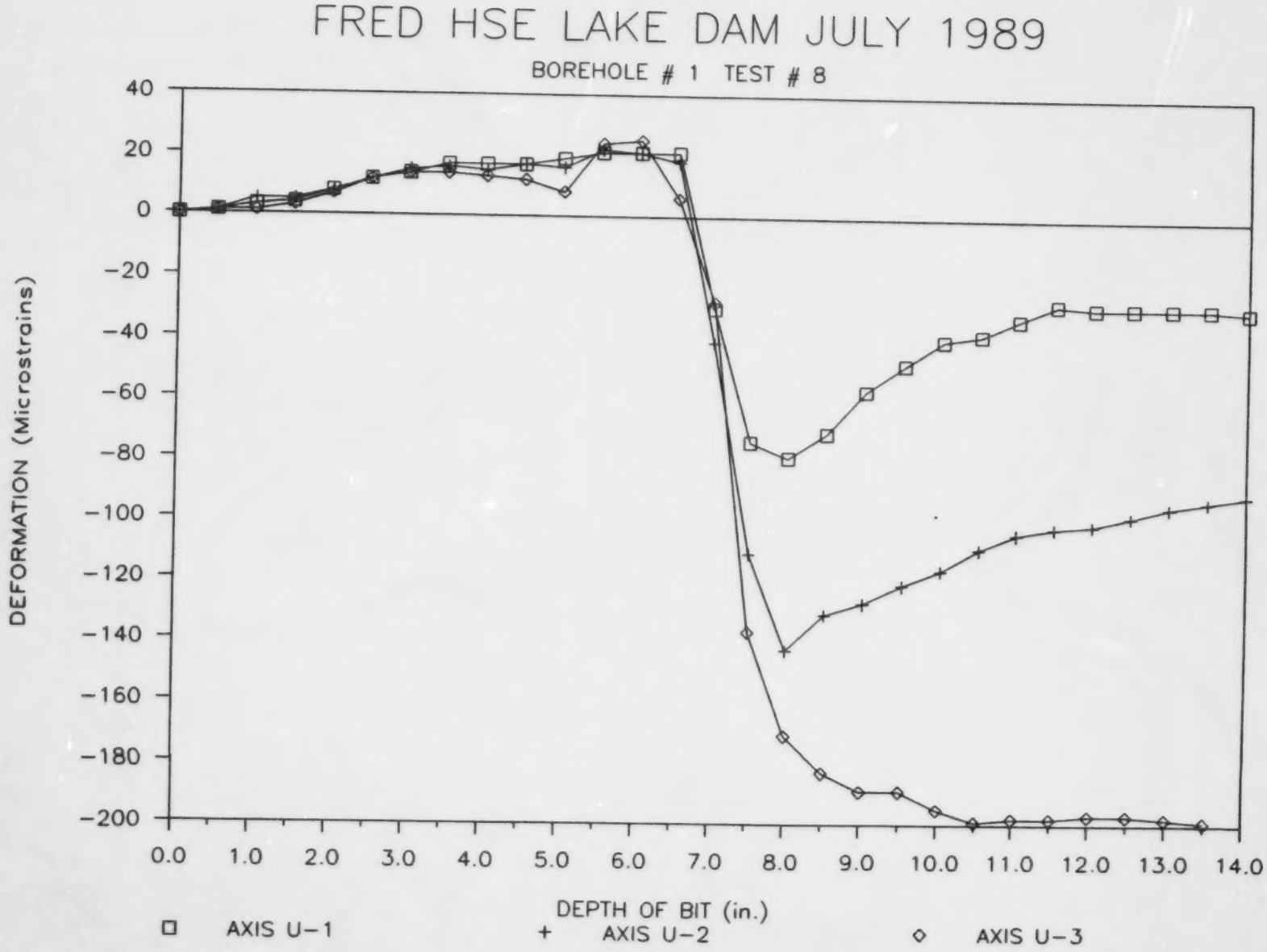

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 3
DEFORMATION (Microstrains)
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

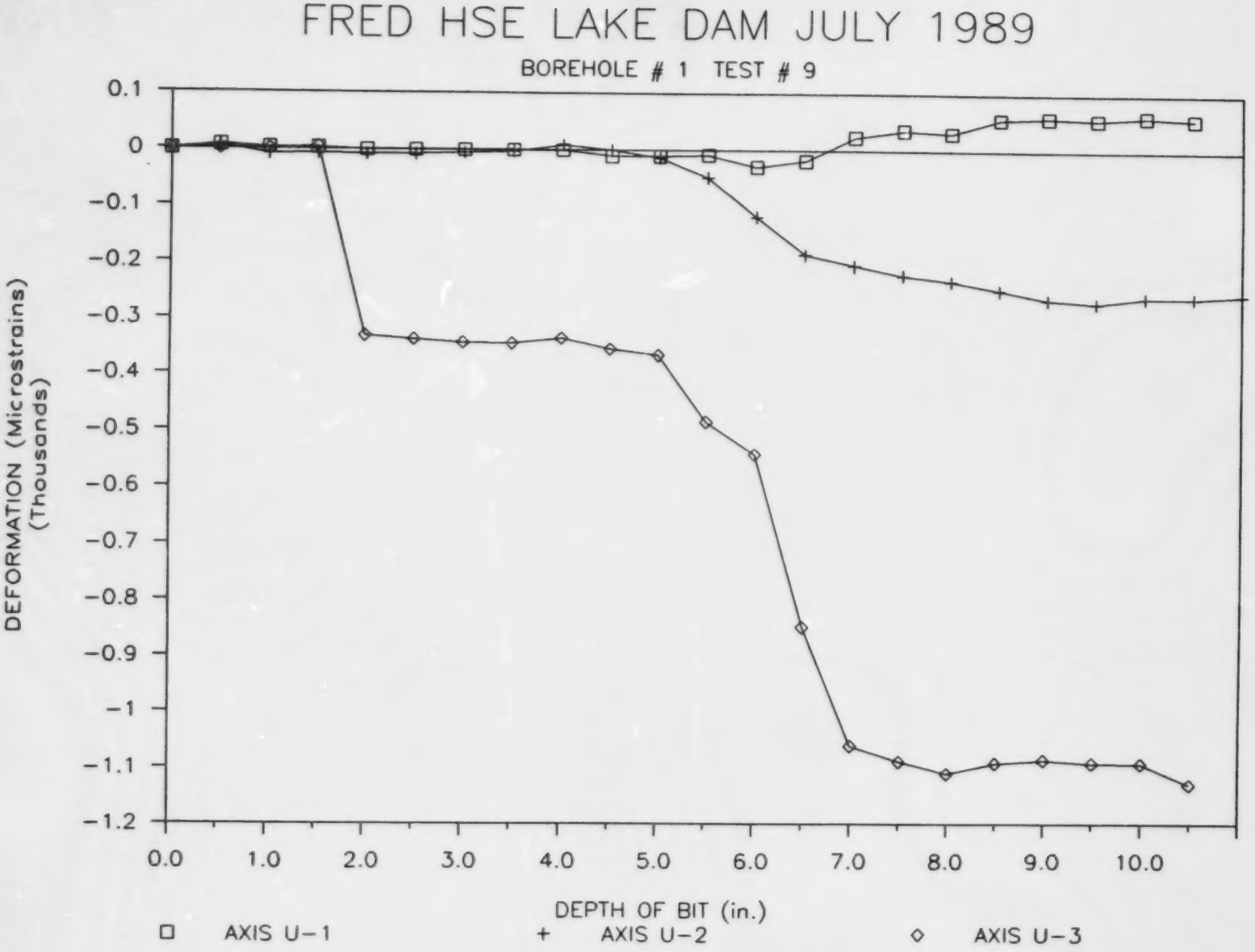

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 4
DEFORMATION (Microstrains)
500
400
300
200
100
0
-100
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

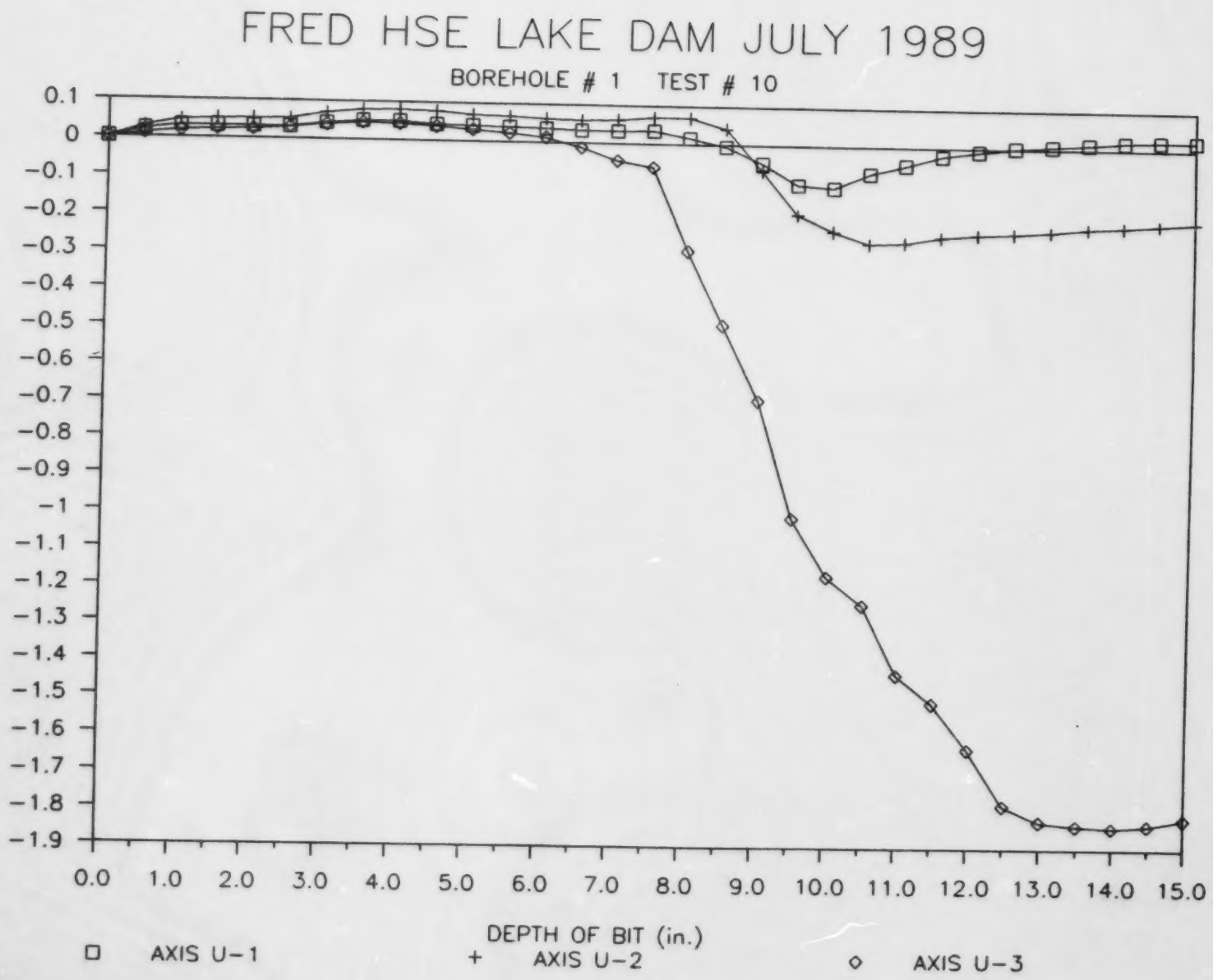

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 5
DEFORMATION (Microstrains)
200
100
0
-100
-200
-300
-400
-500
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

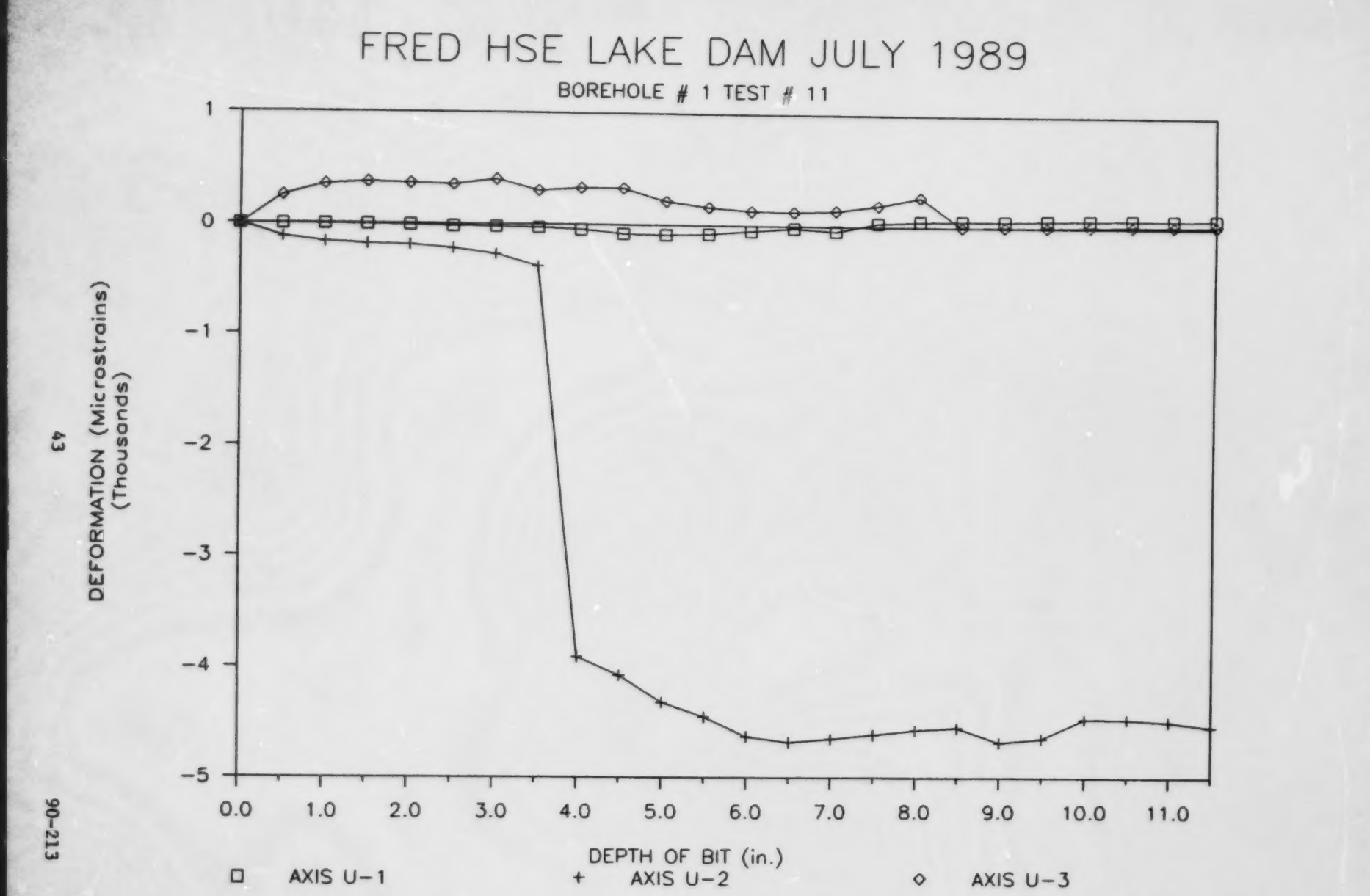

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 6
DEFORMATION (Microstrains)
180
160
140
120
100
80
60
40
20
0
-20
-40
-60
-80
-100
-120
-140
0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0 16.0 17.0 18.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

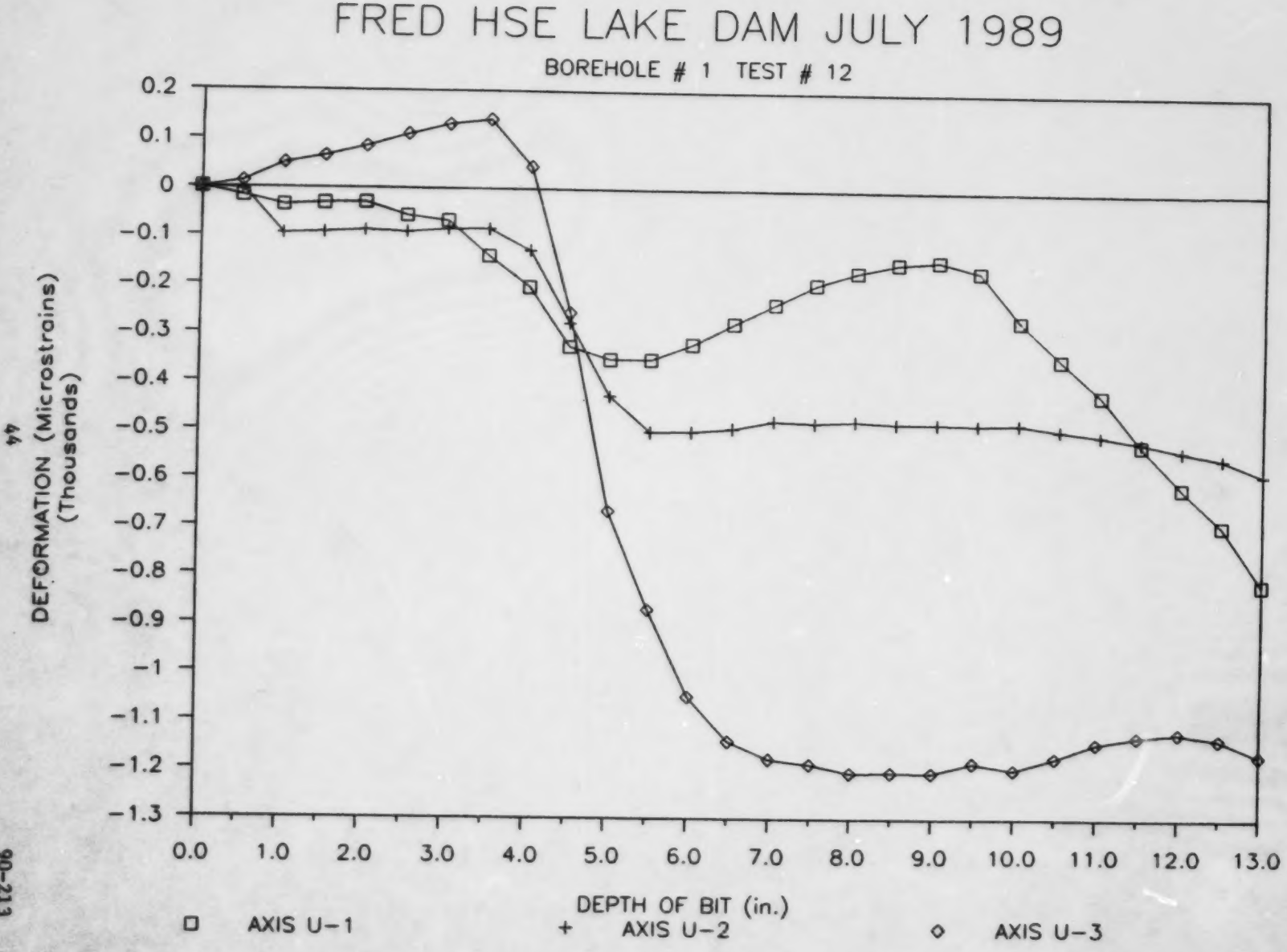

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 7
DEFORMATION (Microstrains)
80
60
40
20
0
-20
-40
-60
-80
-100
-120
-140
-160
-180
-200
0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

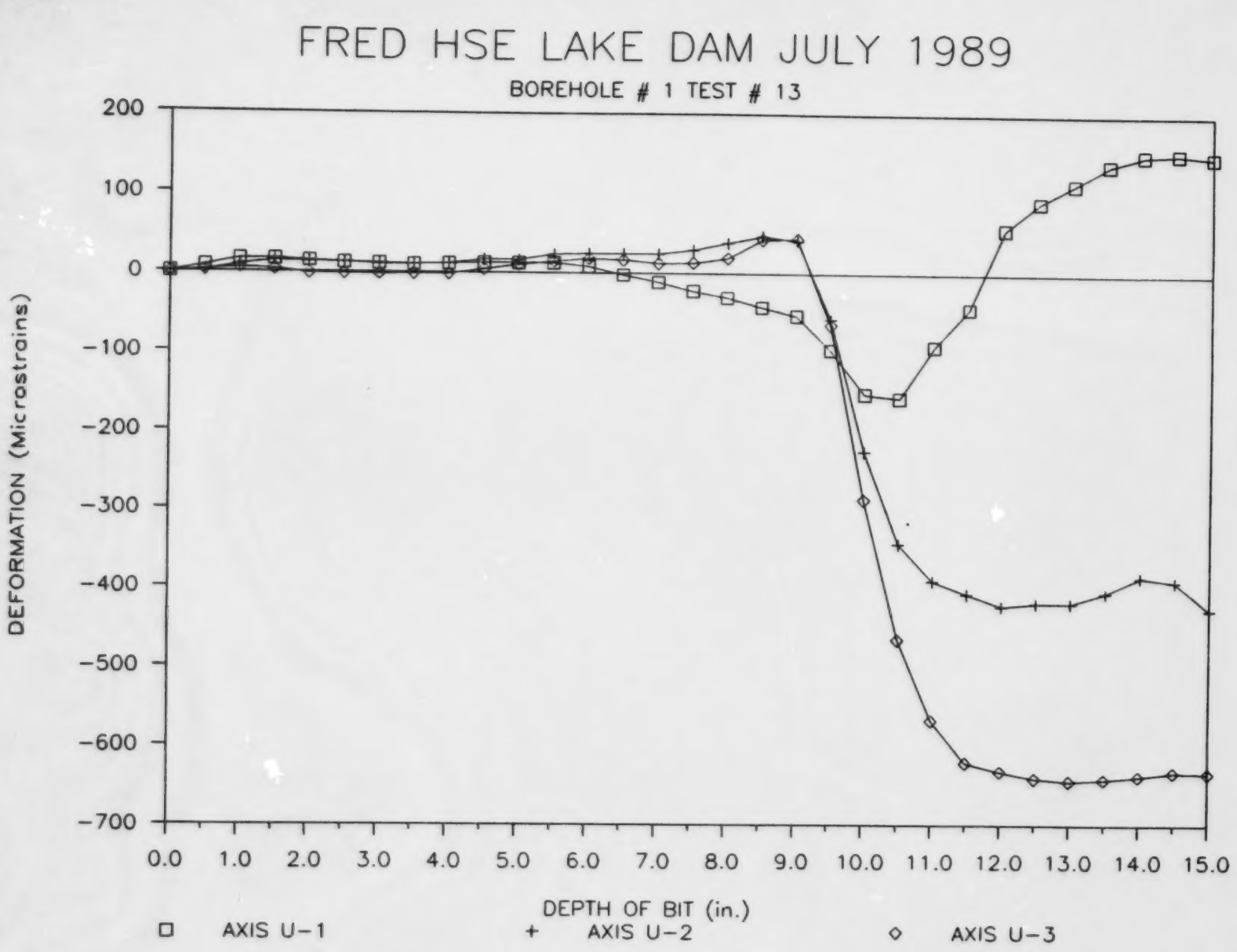

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 8
DEFORMATION (Microstrains)
40
20
0
-20
-40
-60
-80
-100
-120
-140
-160
-180
-200
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

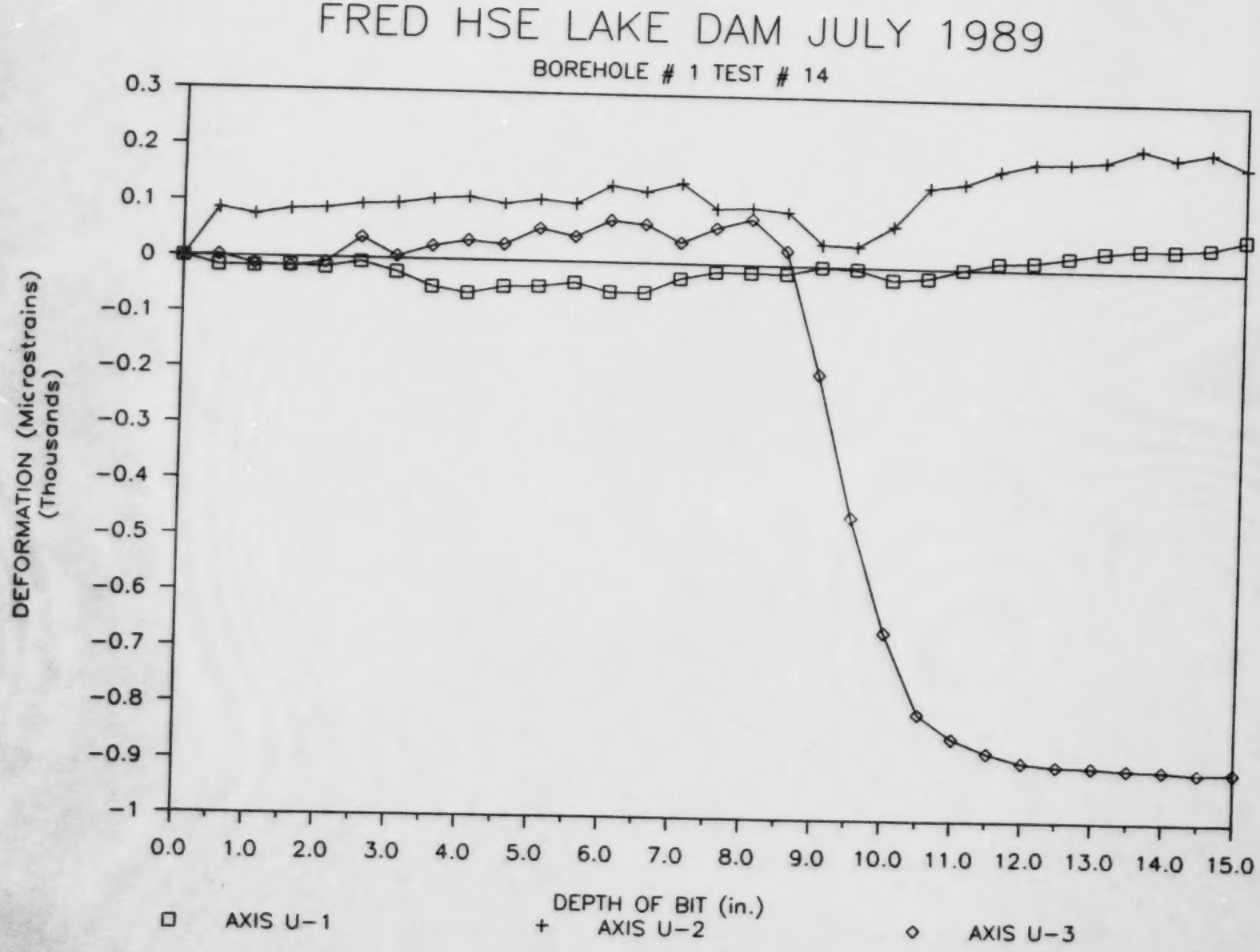

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 9
DEFORMATION (Microstrains)
(Thousands)
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1
-1.1
-1.2
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

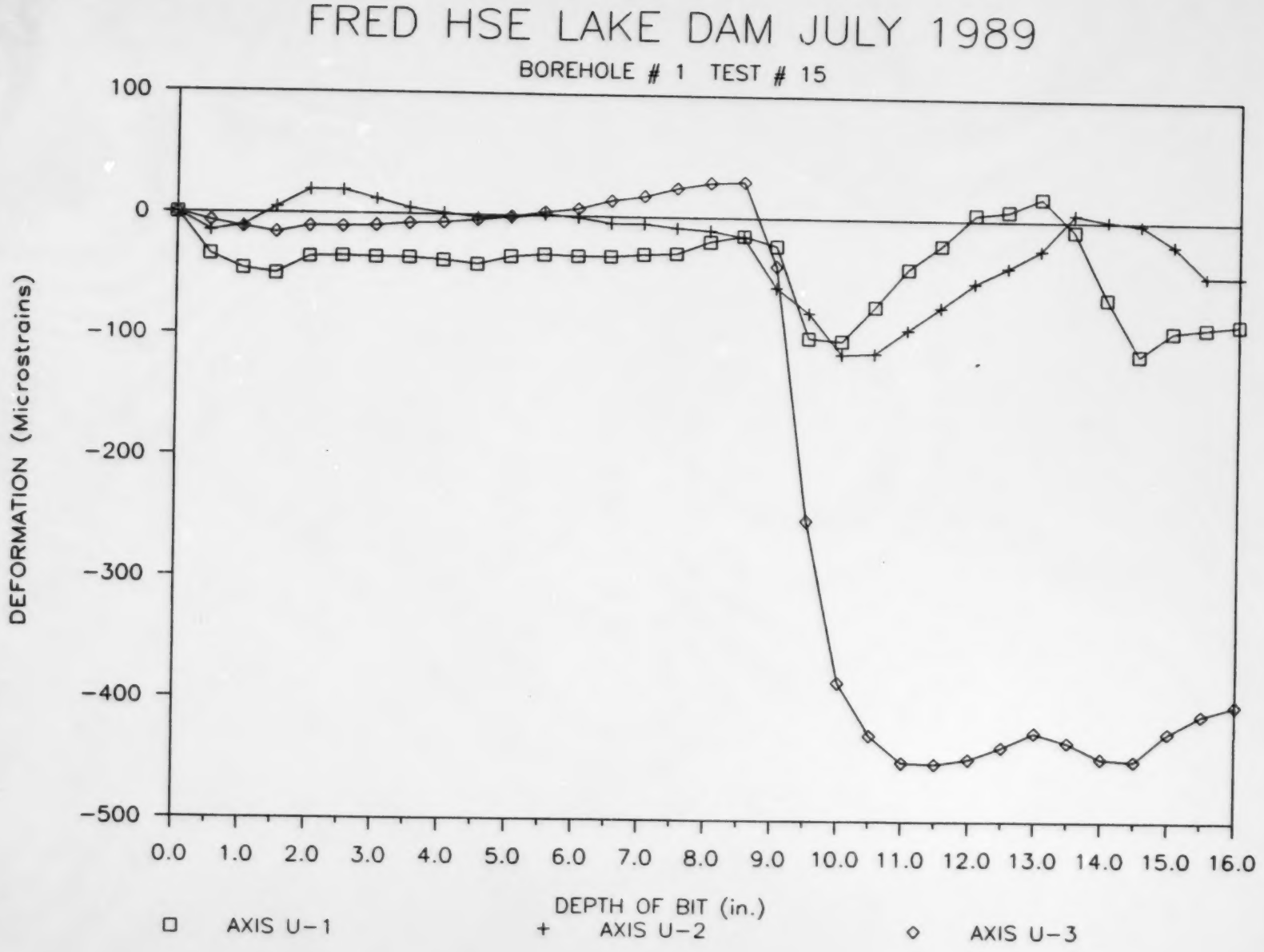

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 10
DEFORMATION (Microstrains)
(Thousands)
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1
-1.1
-1.2
-1.3
-1.4
-1.5
-1.6
-1.7
-1.8
-1.9
0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

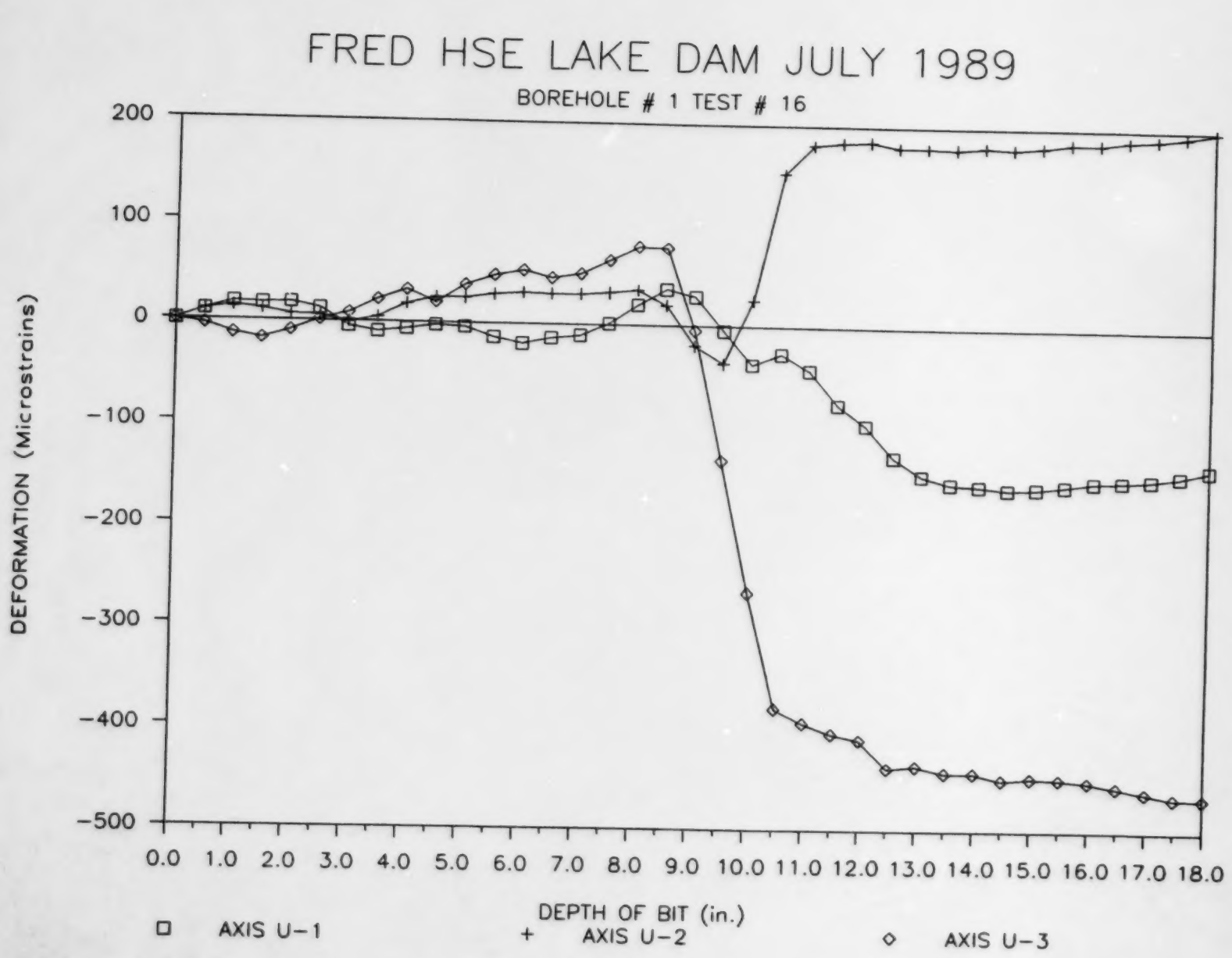

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 11
DEFORMATION (Microstrains)
(Thousands)
1
0
-1
-2
-3
-4
-5
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 12
DEFORMATION (Microstrains)
(Thousands)
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1
-1.1
-1.2
-1.3
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3
44
90-213

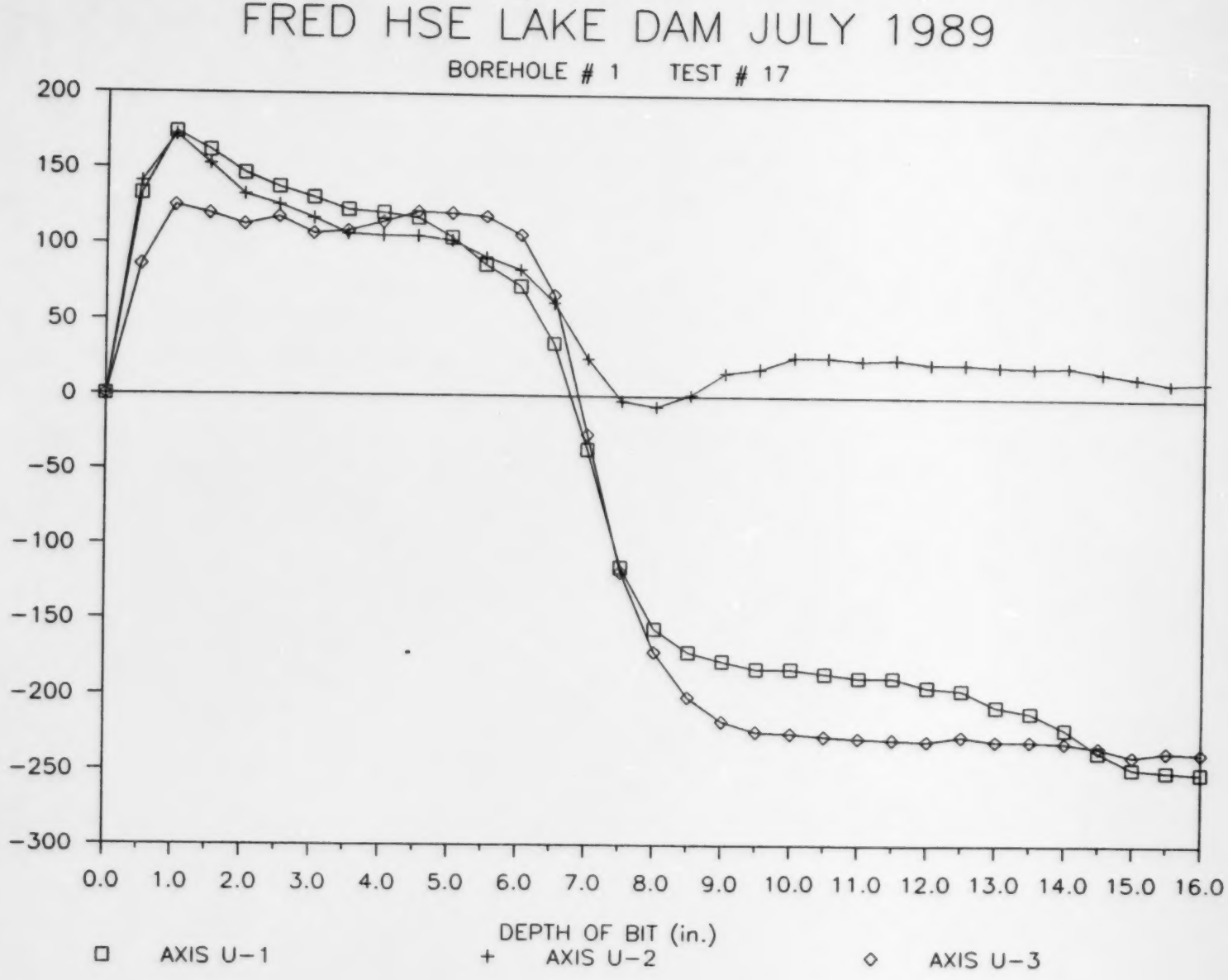

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 13
DEFORMATION (Microstrains)
200
100
0
-100
-200
-300
-400
-500
-600
-700
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

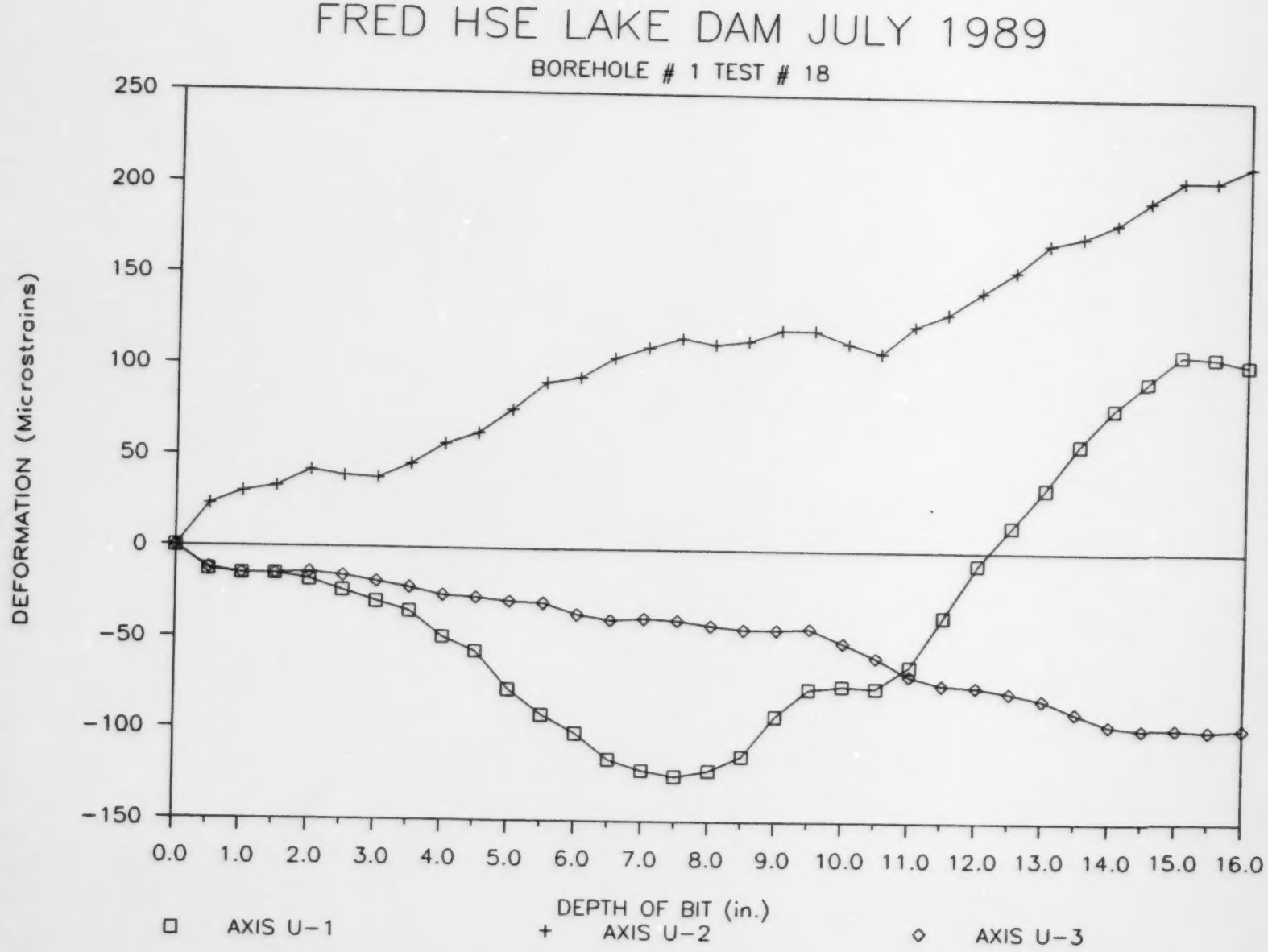

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 14
DEFORMATION (Microstrains)
(Thousands)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 15
100
0
-100
-200
-300
-400
-500
DEFORMATION (Microstrains)
0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0 16.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 16
DEFORMATION (Microstrains)
200
100
0
-100
-200
-300
-400
-500
0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0 16.0 17.0 18.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

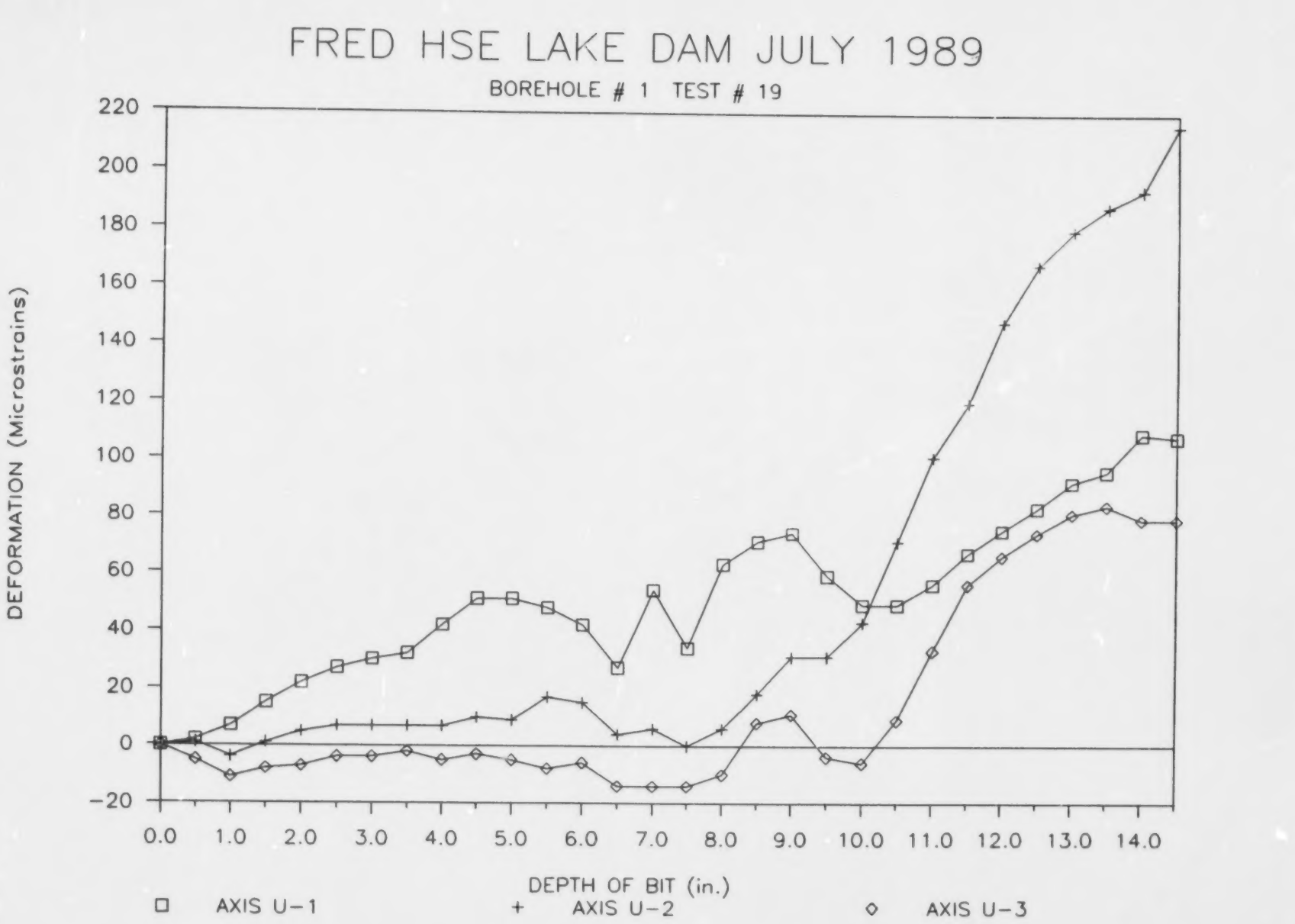
90-213


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 17
DEFORMATION (Microstrains)
200
150
100
50
0
-50
-100
-150
-200
-250
-300
0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0 16.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 18
DEFORMATION (Microstrains)
250
200
150
100
50
0
-50
-100
-150
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

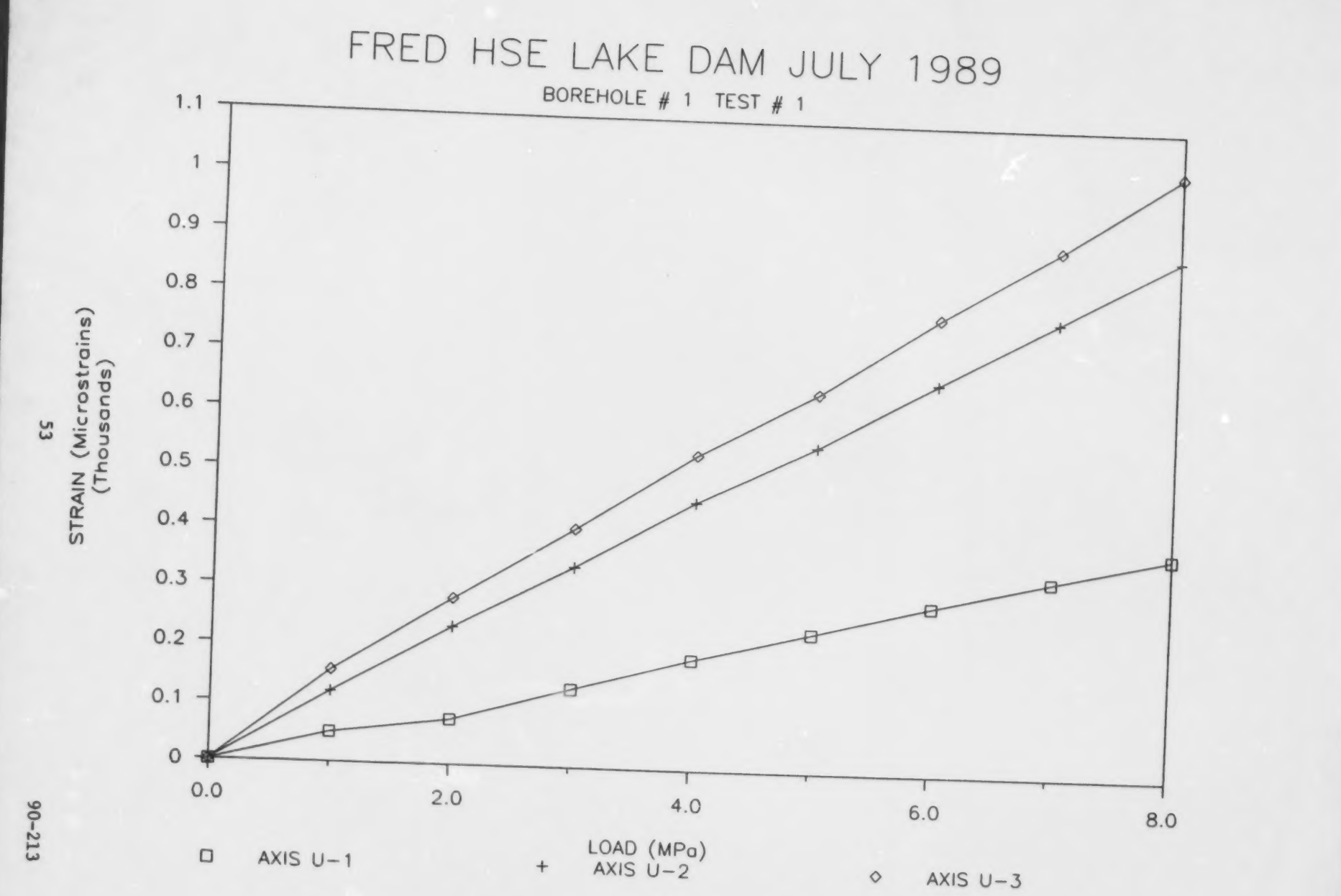
50

90-213


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 19
DEFORMATION (Microstrains)
220
200
180
160
140
120
100
80
60
40
20
0
-20
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
DEPTH OF BIT (in.)
AXIS U-1
AXIS U-2
AXIS U-3

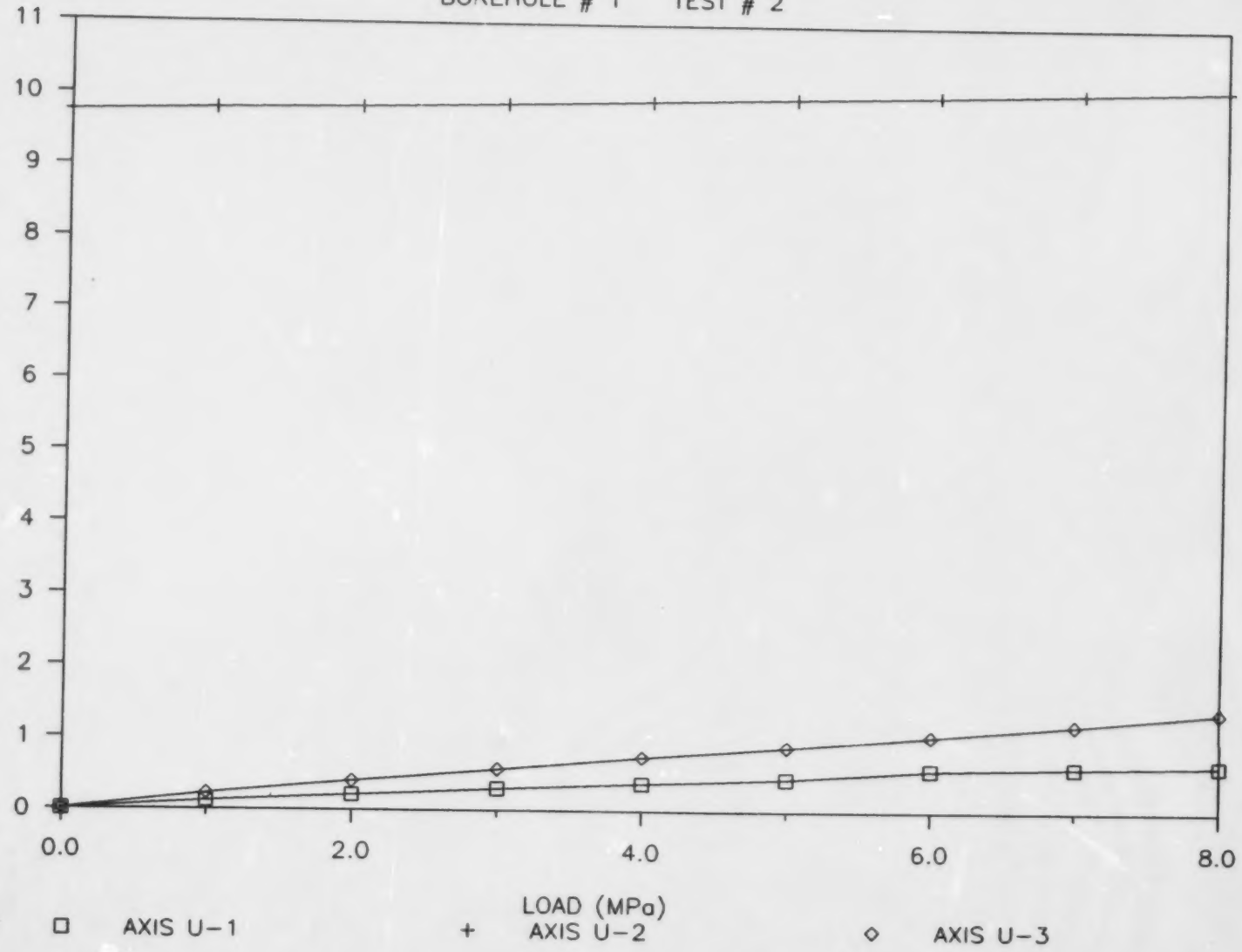
APPENDIX 3

BIAXIAL TEST RESULTS


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 1
STRAIN (Microstrains)
(Thousands)
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0.0
2.0
4.0
6.0
8.0
LOAD (MPa)
AXIS U-1
AXIS U-2
AXIS U-3

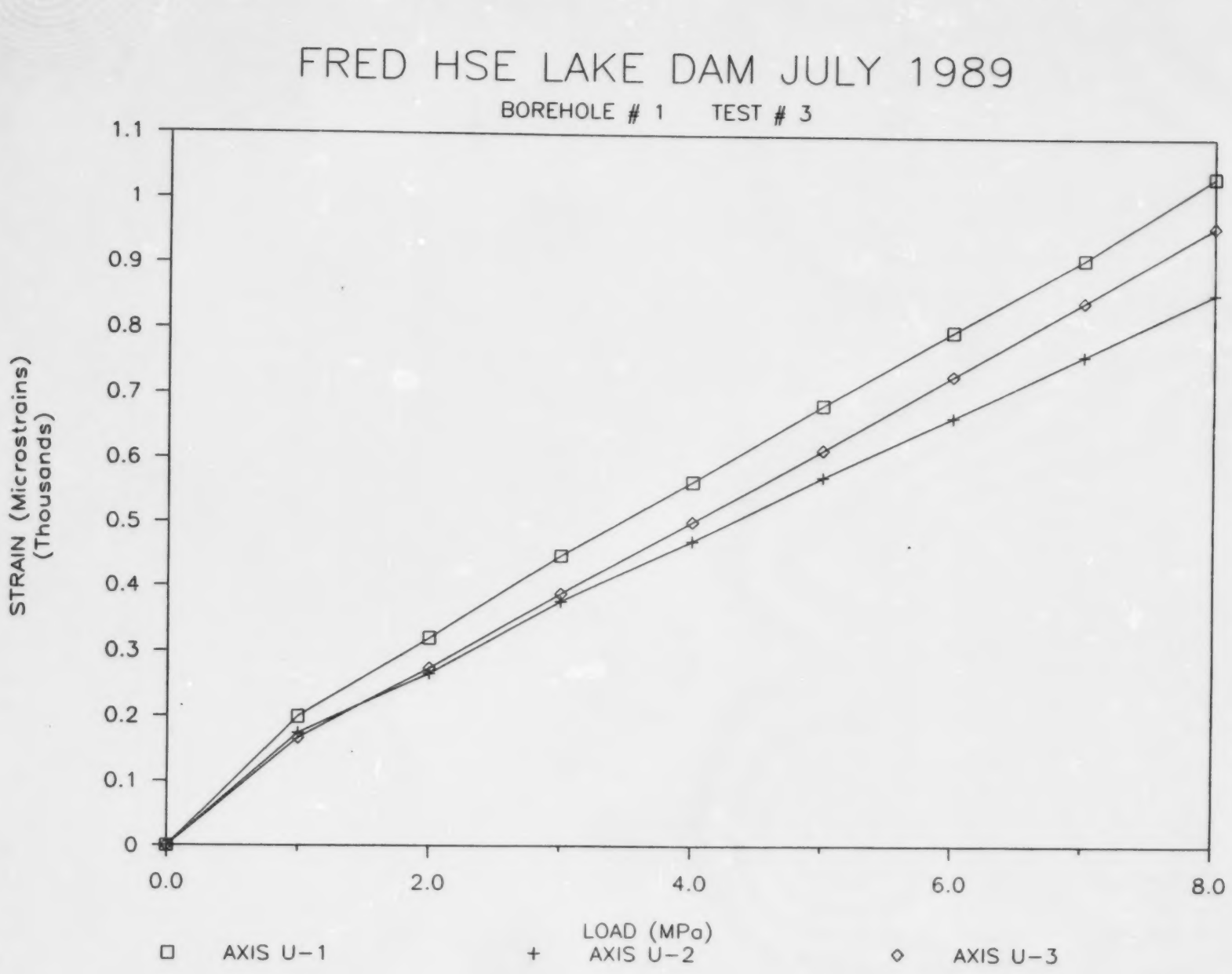
FRED HSE LAKE DAM JULY 1989

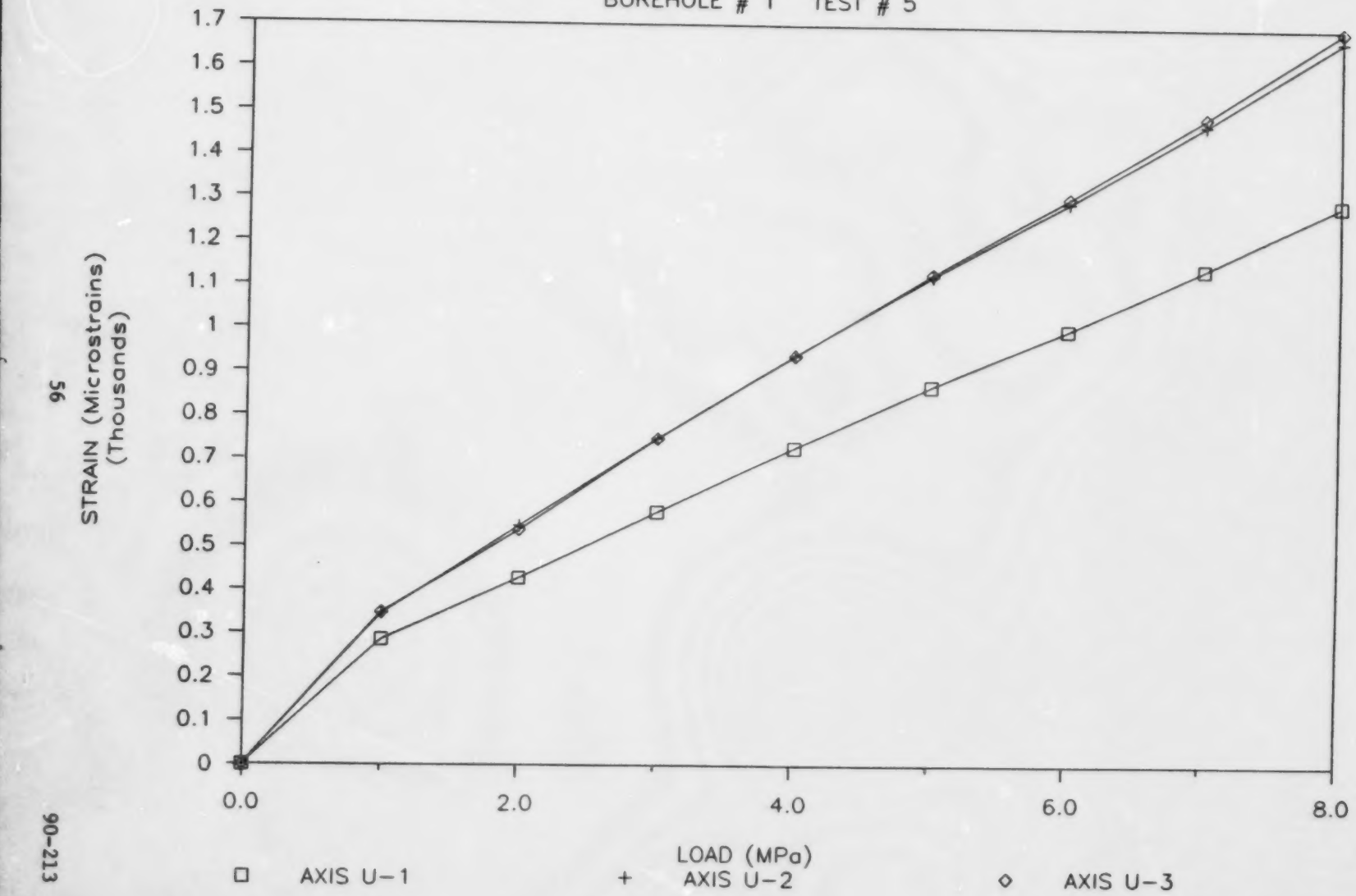
BOREHOLE # 1 TEST # 2

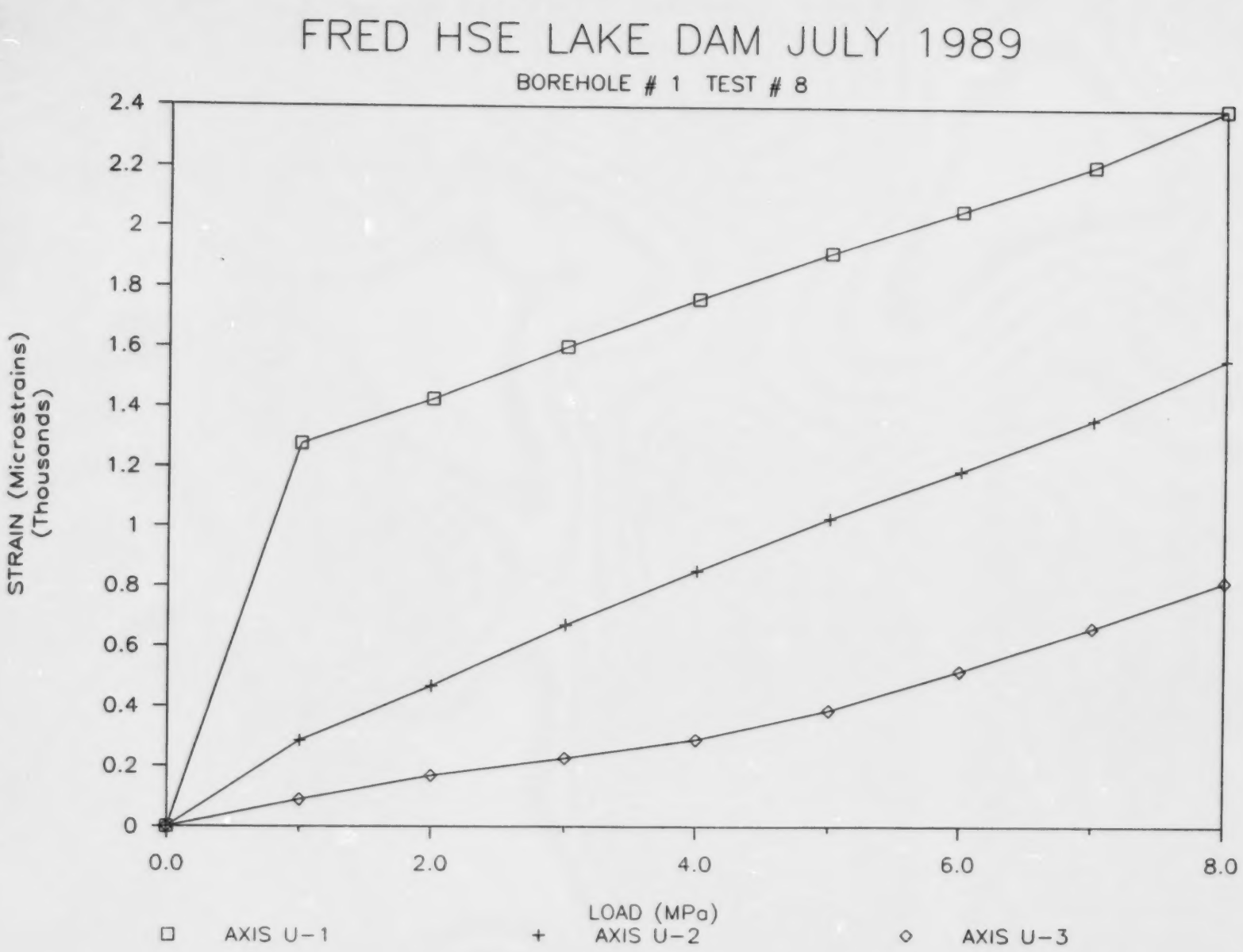


54

90-213

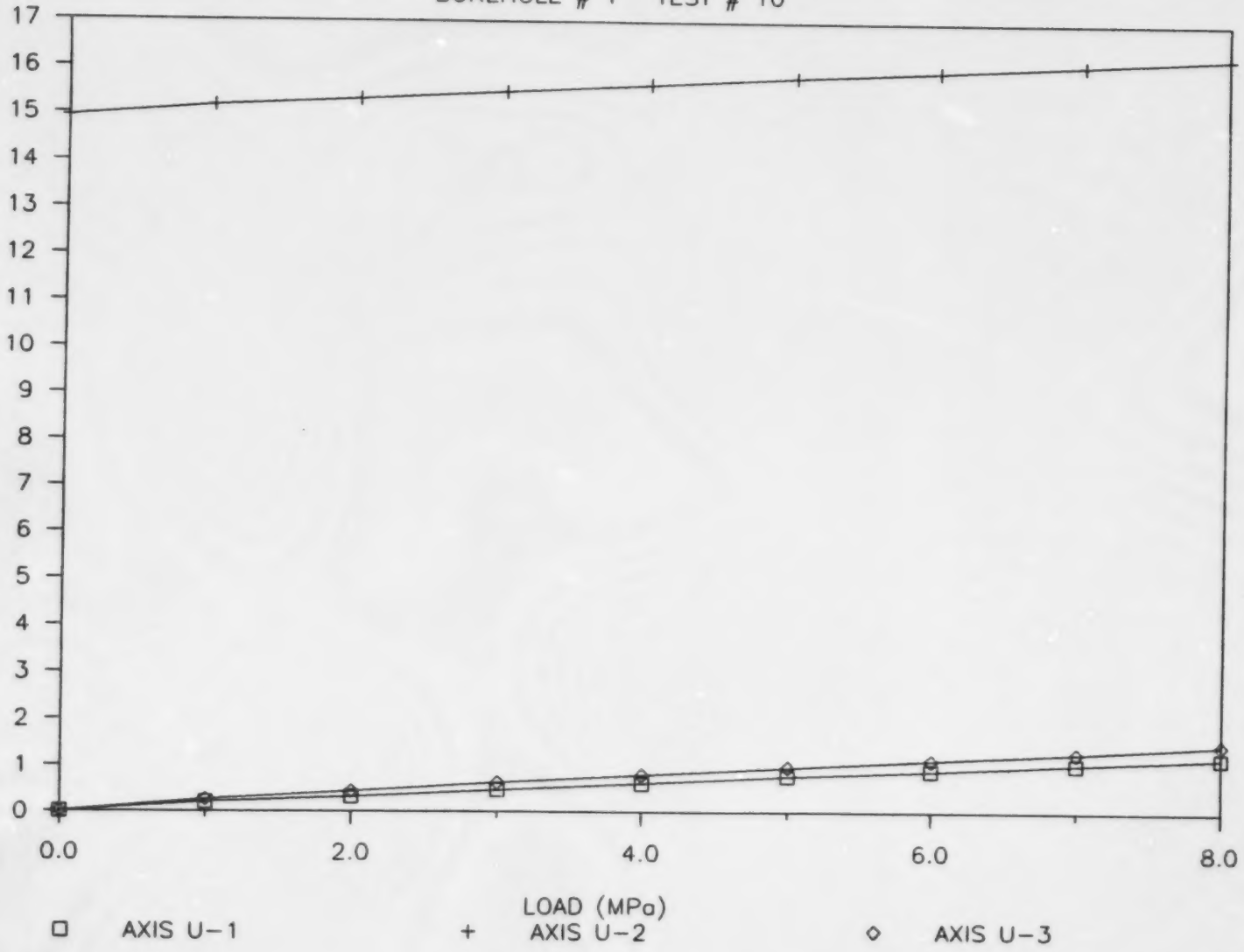

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 3
STRAIN (Microstrains)
(Thousands)
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0.0
2.0
4.0
6.0
8.0
LOAD (MPa)
AXIS U-1
AXIS U-2
AXIS U-3


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 5
STRAIN (Microstrains)
(Thousands)
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0.0
2.0
4.0
6.0
8.0
LOAD (MPa)
AXIS U-1
AXIS U-2
AXIS U-3


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 8
STRAIN (Microstrains)
(Thousands)
2.4
2.2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.0
2.0
4.0
6.0
8.0
LOAD (MPa)
AXIS U-1
AXIS U-2
AXIS U-3

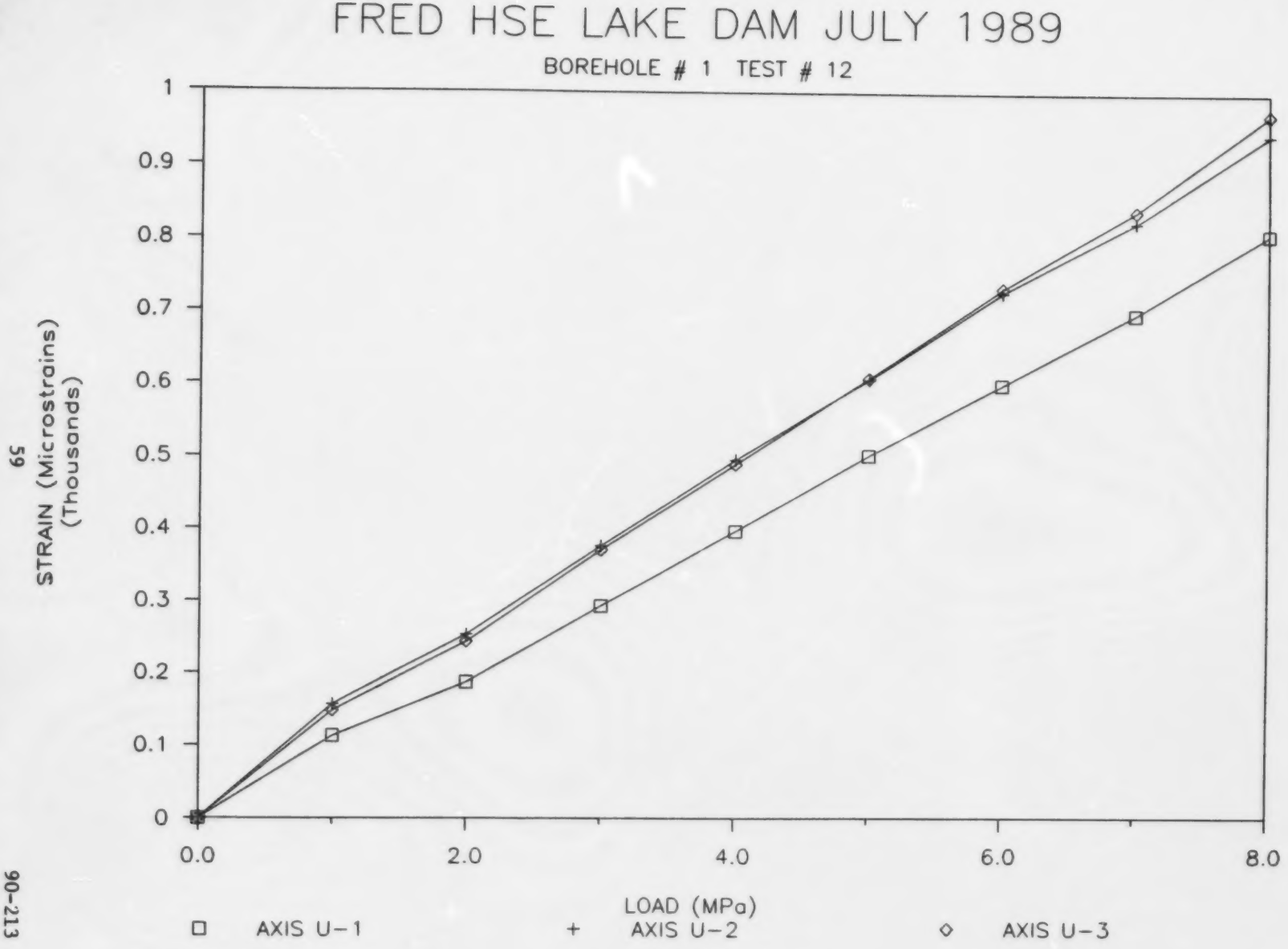
FRED HSE LAKE DAM JULY 1989

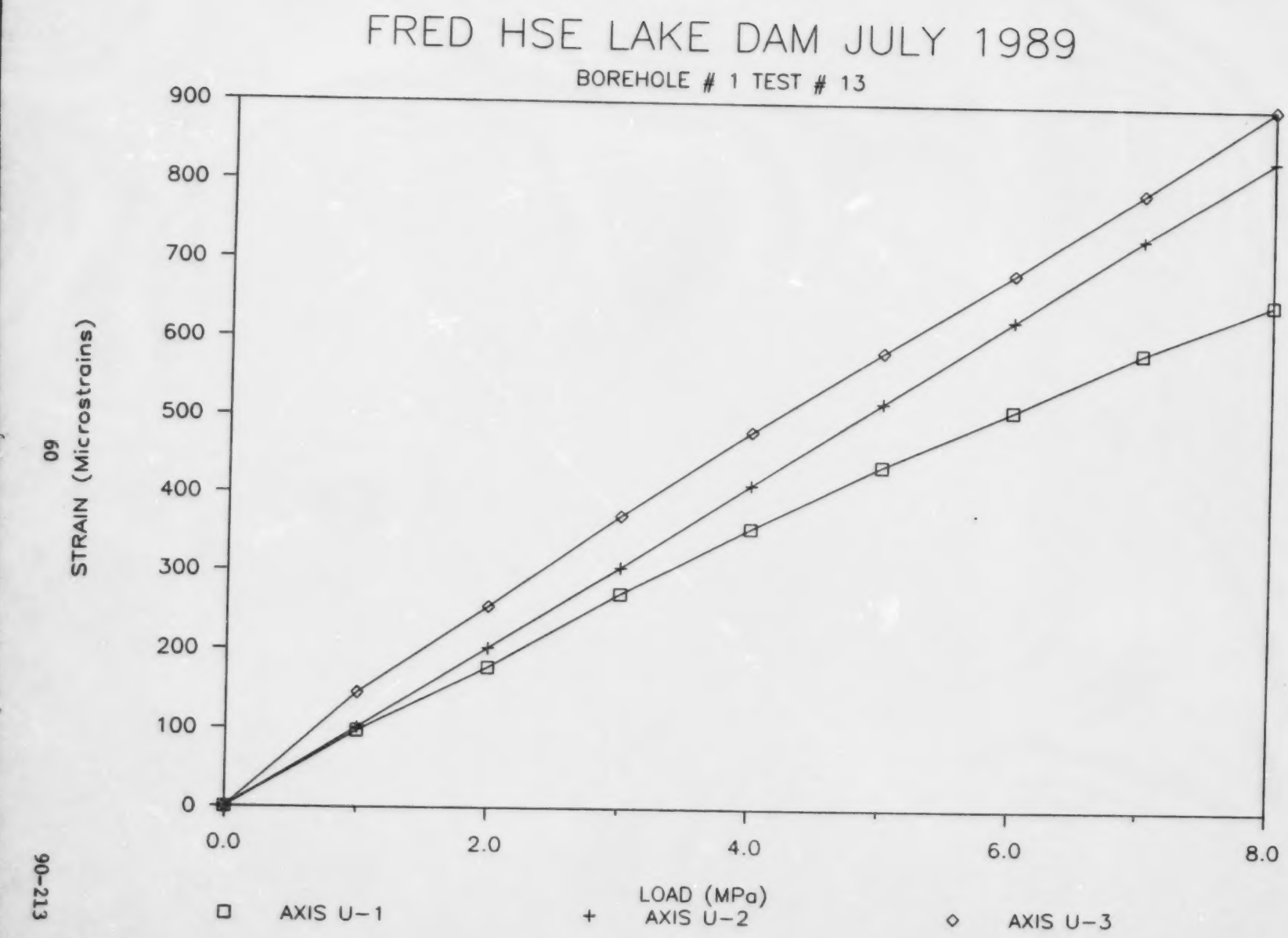
BOREHOLE # 1 TEST # 10



58

90-213


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 12
STRAIN (Microstrains)
(Thousands)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0.0
2.0
4.0
6.0
8.0
LOAD (MPa)
AXIS U-1
AXIS U-2
AXIS U-3

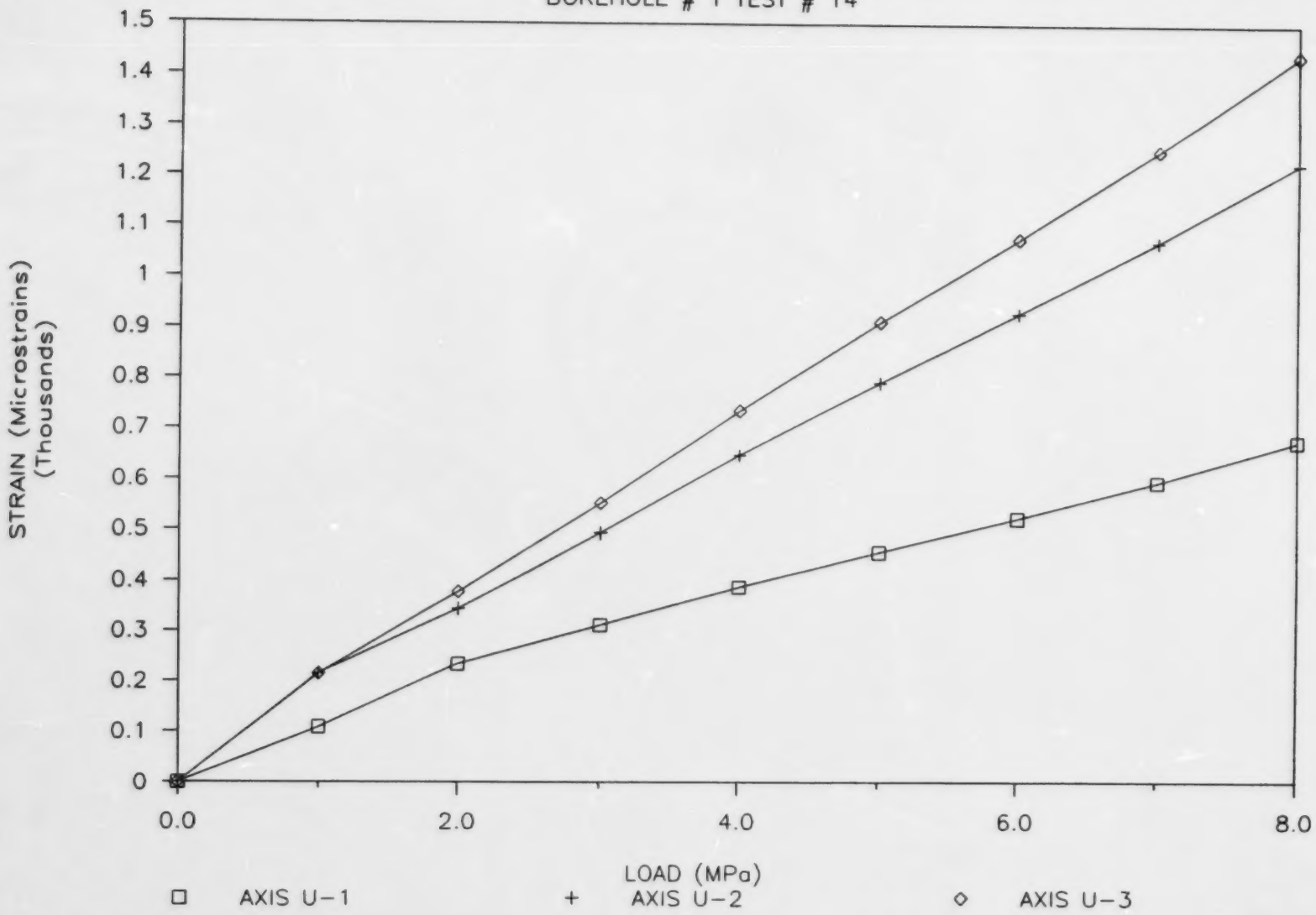

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 13
STRAIN (Microstrains)
900
800
700
600
500
400
300
200
100
0
0.0
2.0
4.0
6.0
8.0
LOAD (MPa)
AXIS U-1
AXIS U-2
AXIS U-3

60

90-213

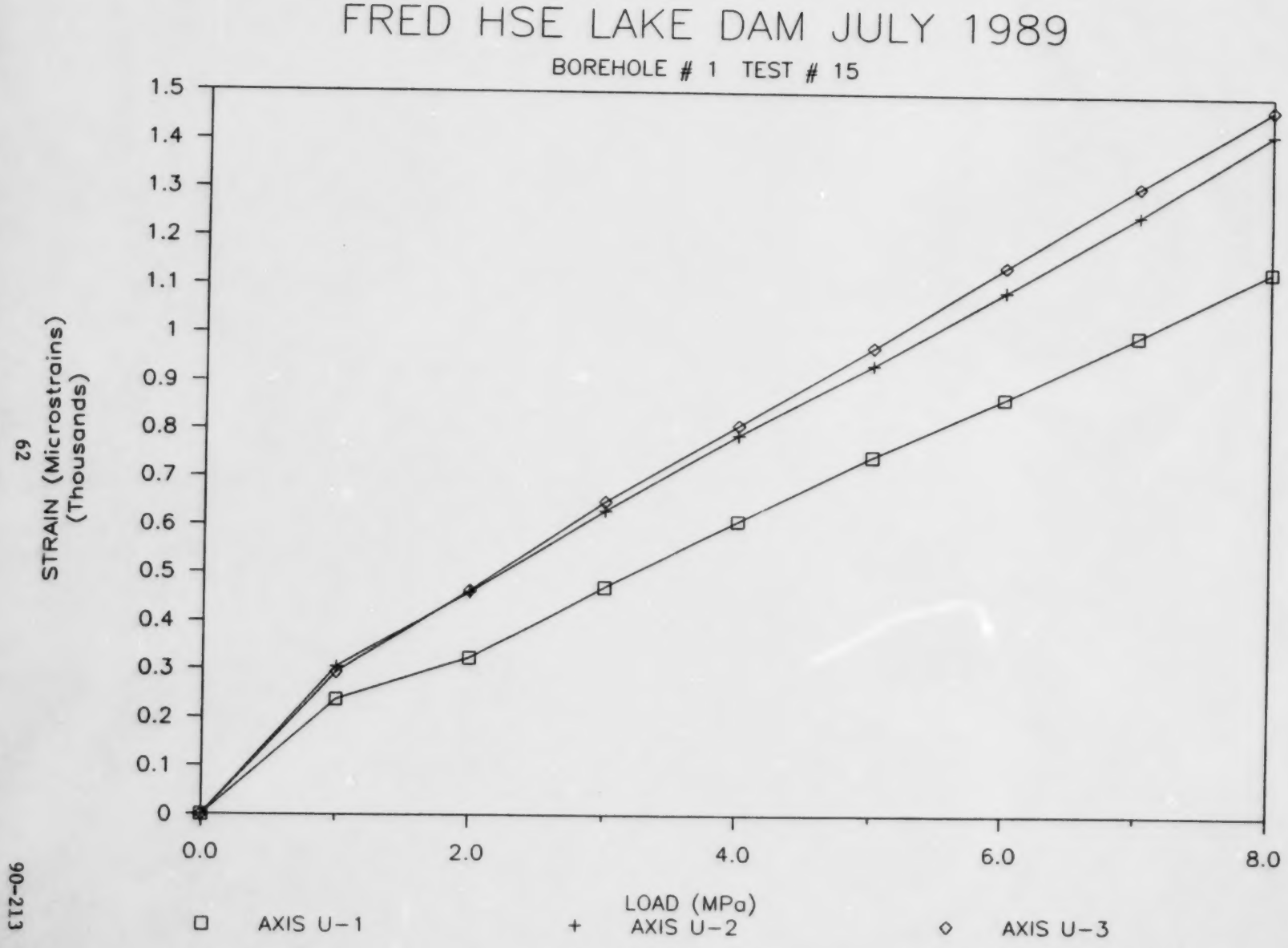
FRED HSE LAKE DAM JULY 1989

BOREHOLE # 1 TEST # 14



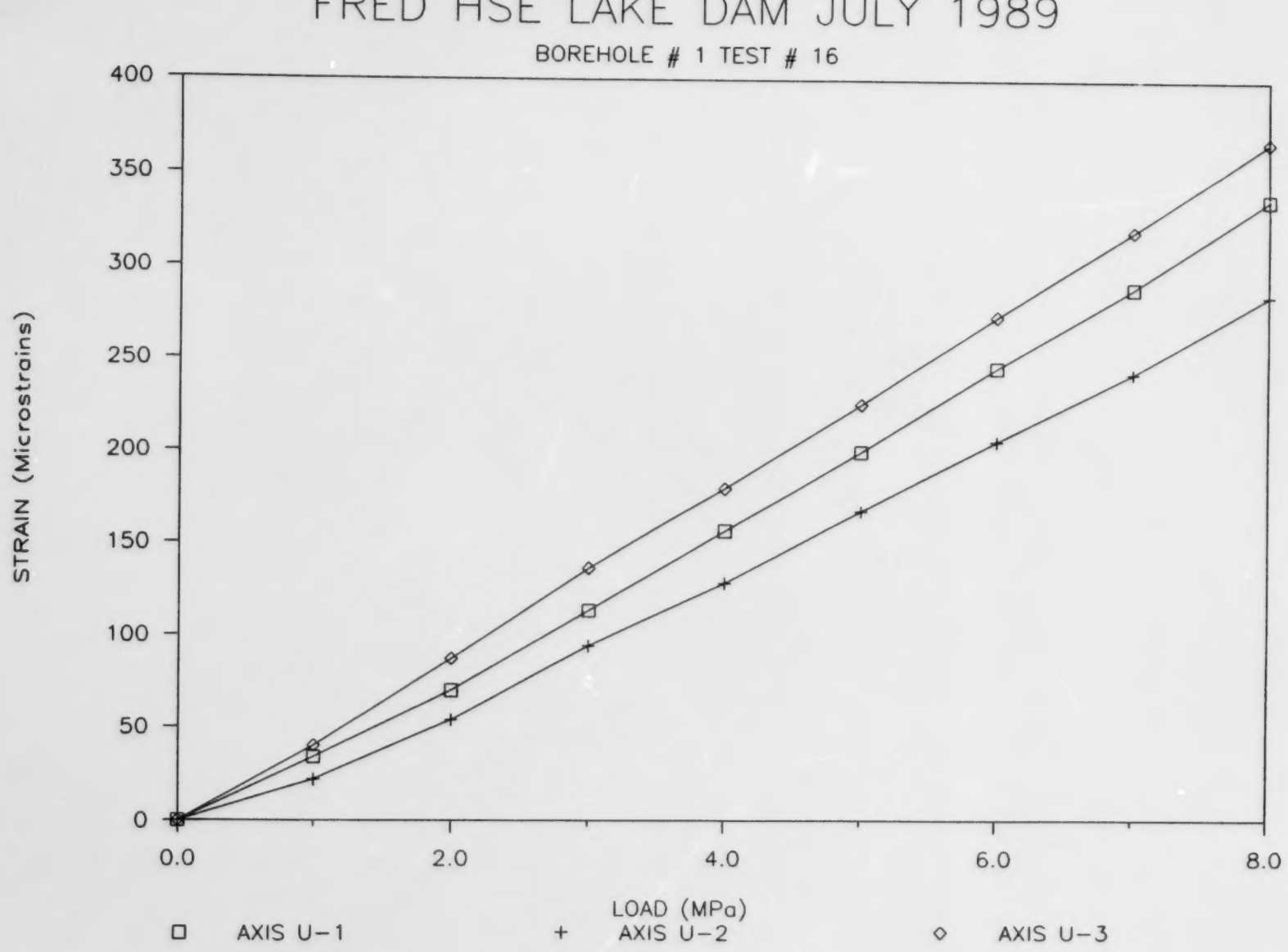
T9 61

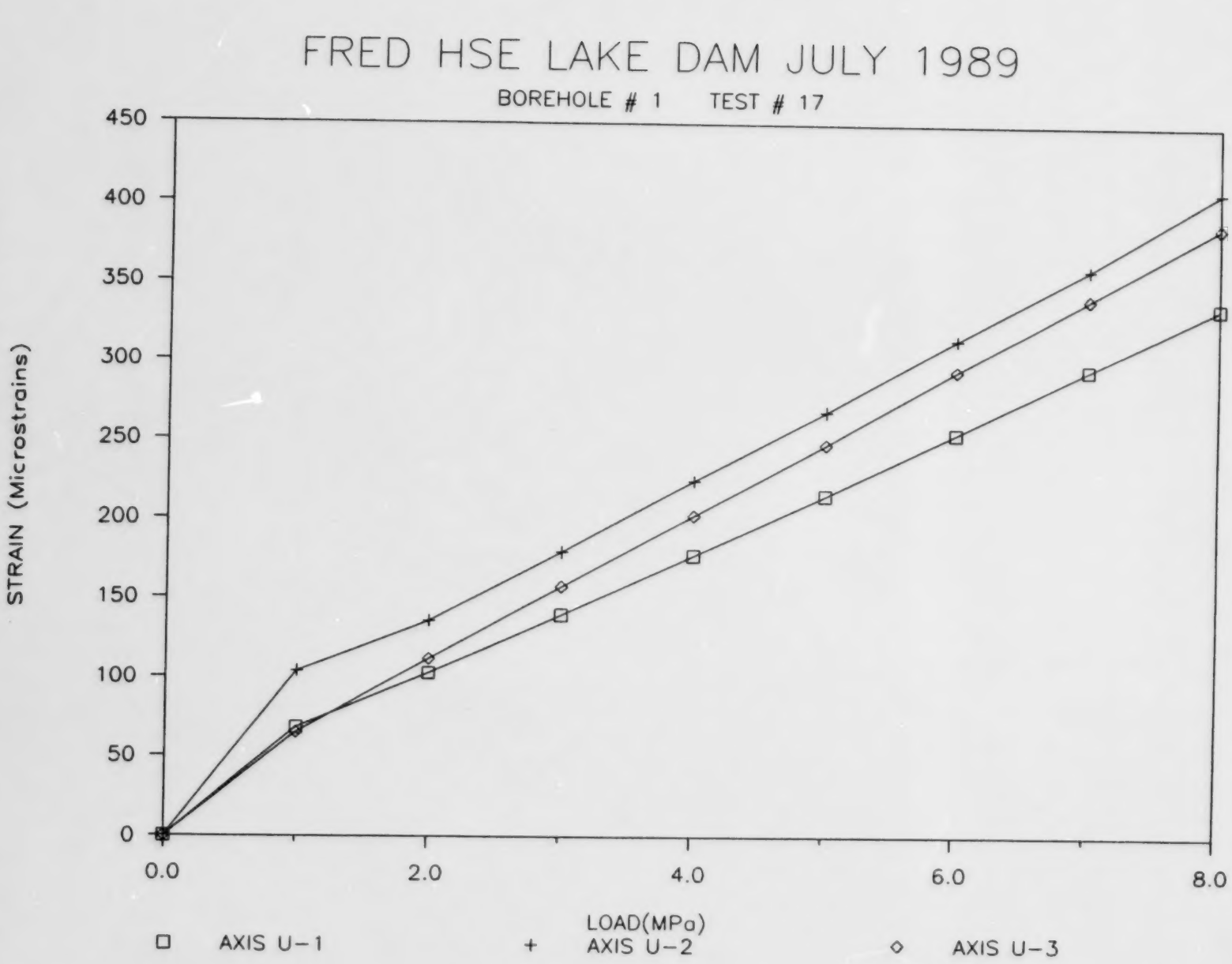
90-213


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 15
STRAIN (Microstrains)
(Thousands)
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0.0
2.0
4.0
6.0
8.0
LOAD (MPa)
AXIS U-1
AXIS U-2
AXIS U-3

62

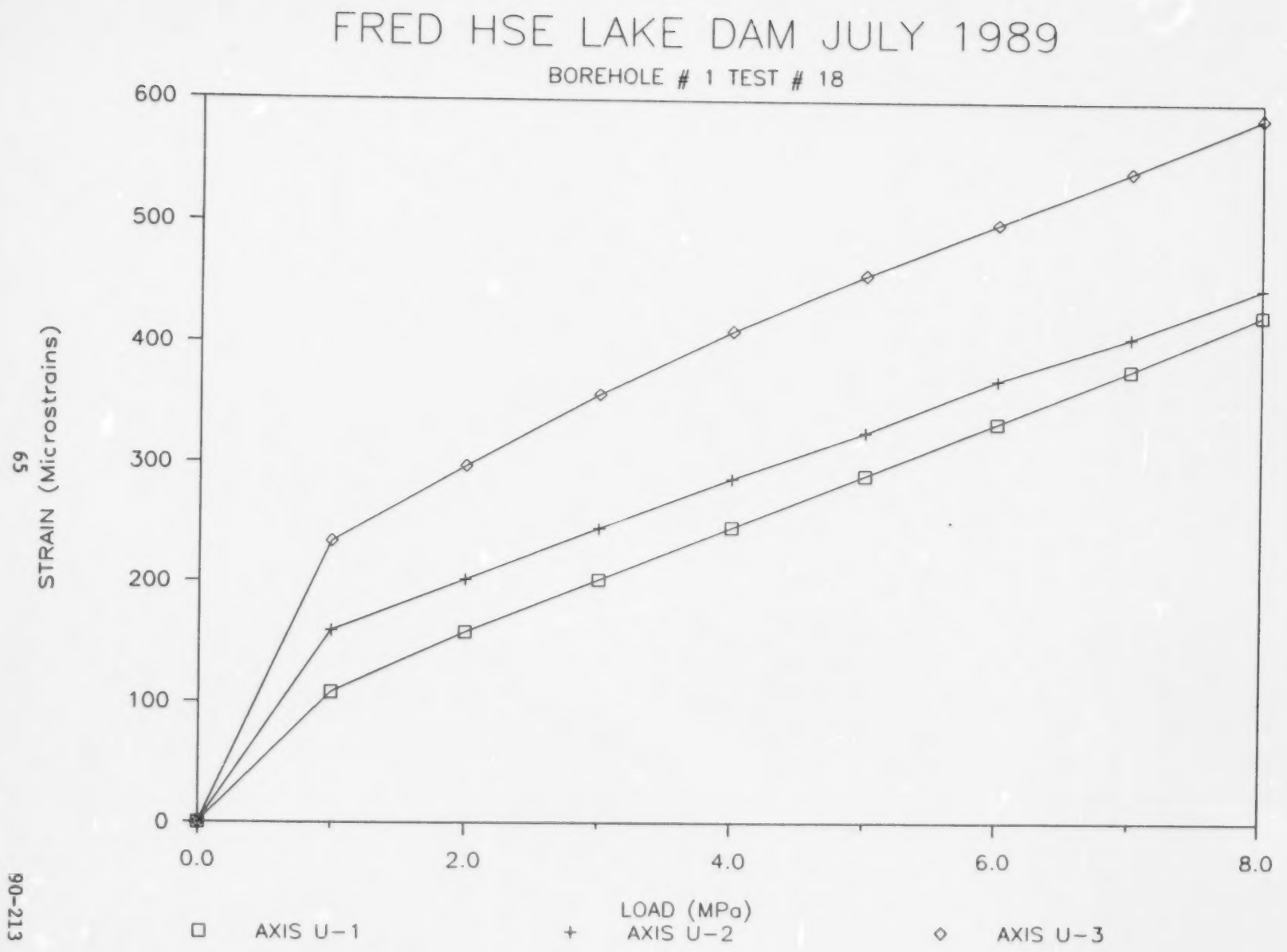
90-213

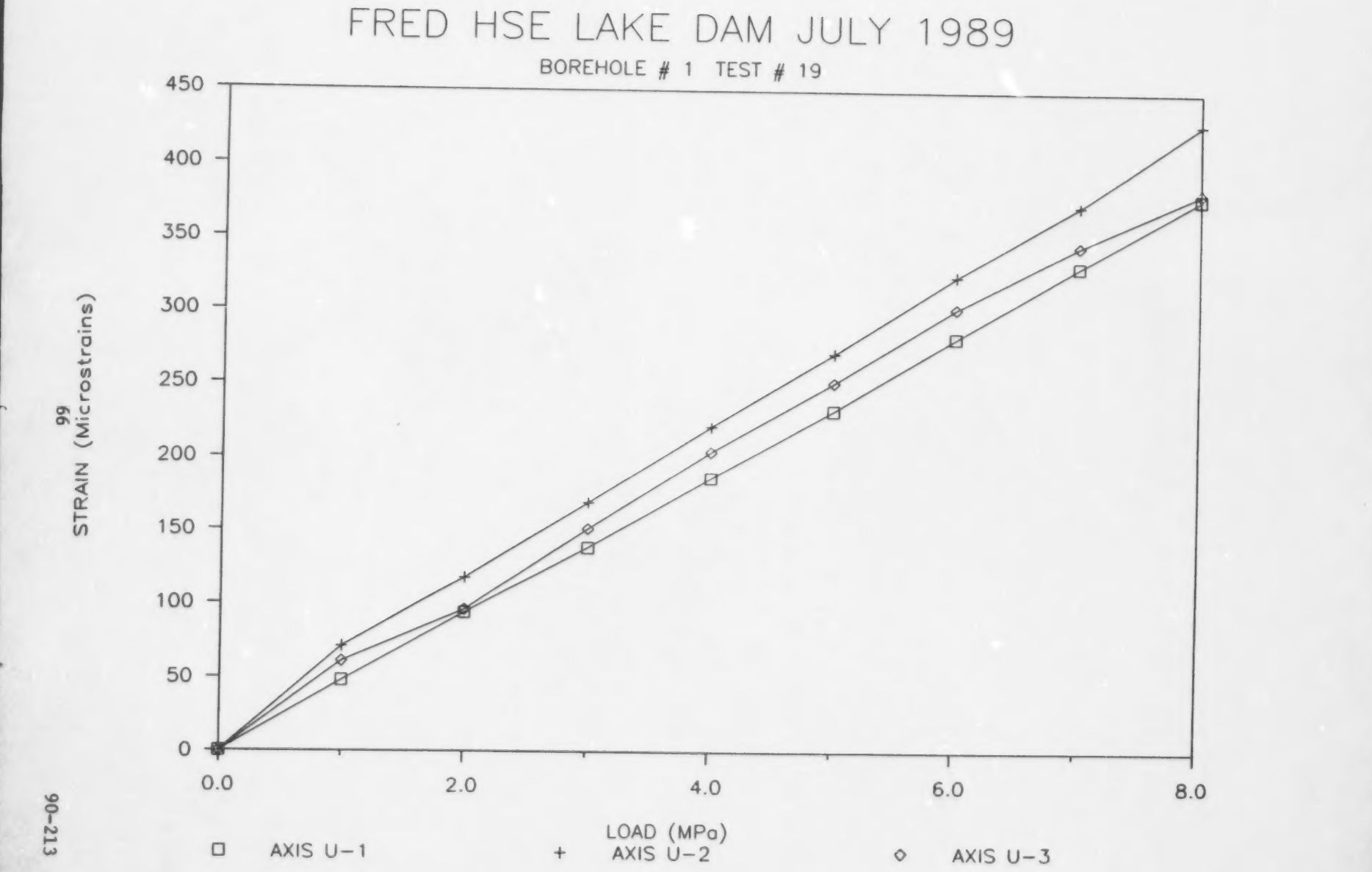

FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 16
400
350
300
250
200
150
100
50
0
STRAIN (Microstrains)
0.0
2.0
4.0
6.0
8.0
LOAD (MPa)
AXIS U-1
AXIS U-2
AXIS U-3


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 17
STRAIN (Microstrains)
450
400
350
300
250
200
150
100
50
0
0.0
2.0
4.0
6.0
8.0
LOAD(MPa)
AXIS U-1
AXIS U-2
AXIS U-3

64

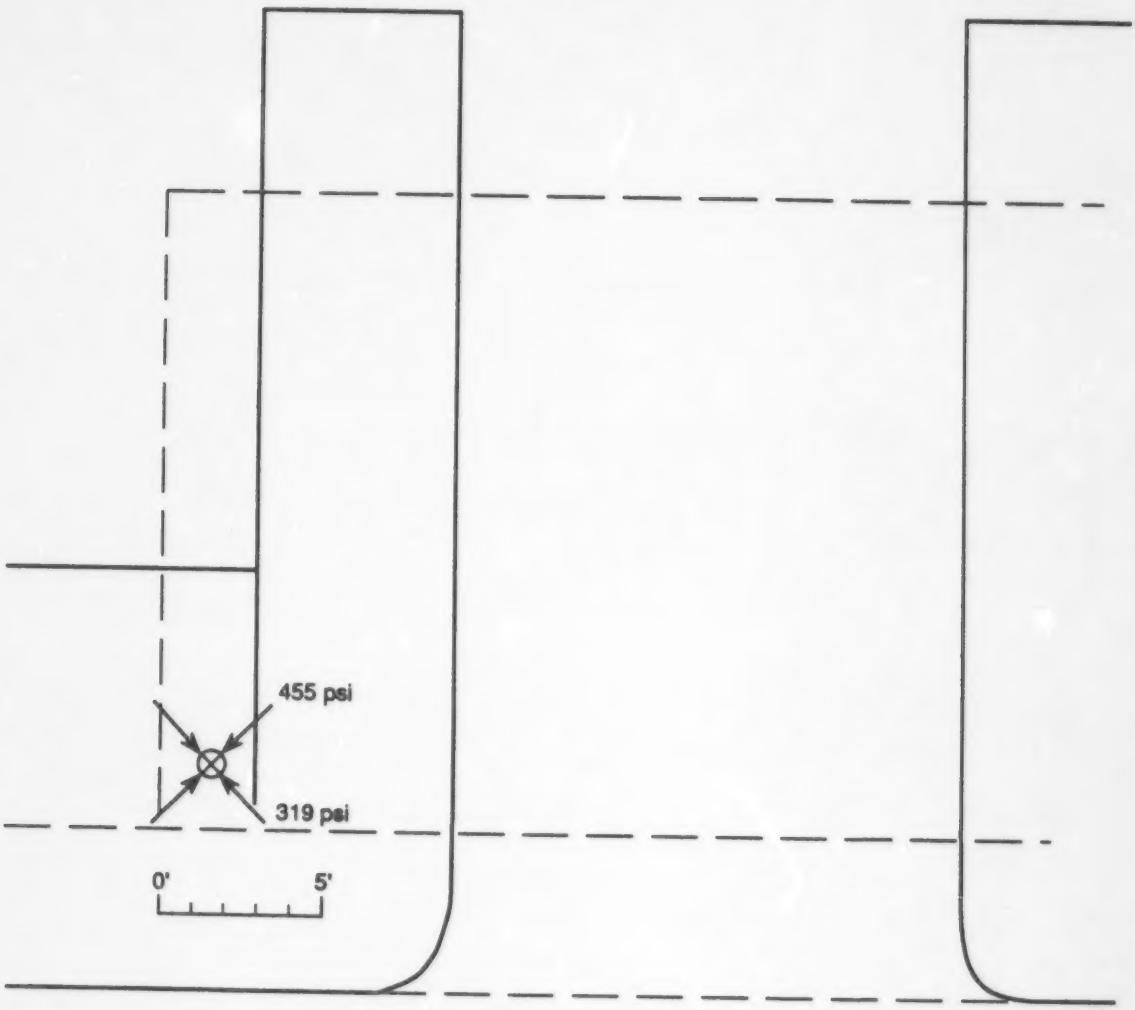
90-213


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 18
600
500
400
300
200
100
0
STRAIN (Microstrains)
0.0
2.0
4.0
6.0
8.0
LOAD (MPa)
AXIS U-1
AXIS U-2
AXIS U-3


FRED HSE LAKE DAM JULY 1989
BOREHOLE # 1 TEST # 19
STRAIN (Microstrains)
99
450
400
350
300
250
200
150
100
50
0
0.0
2.0
4.0
6.0
8.0
LOAD (MPa)
AXIS U-1
AXIS U-2
AXIS U-3
90-213

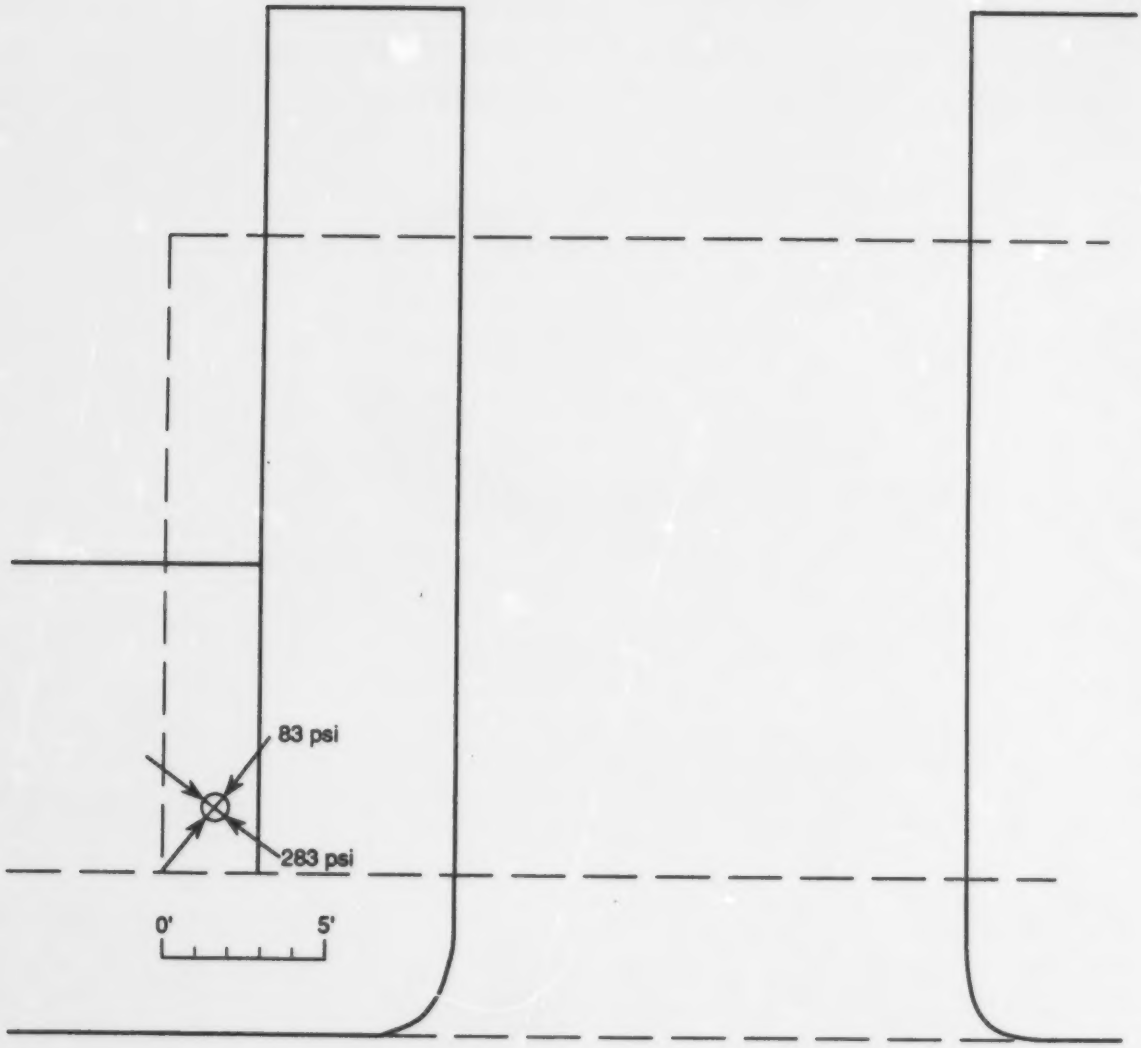
APPENDIX 4

TEST RESULTS - PLAN VIEW



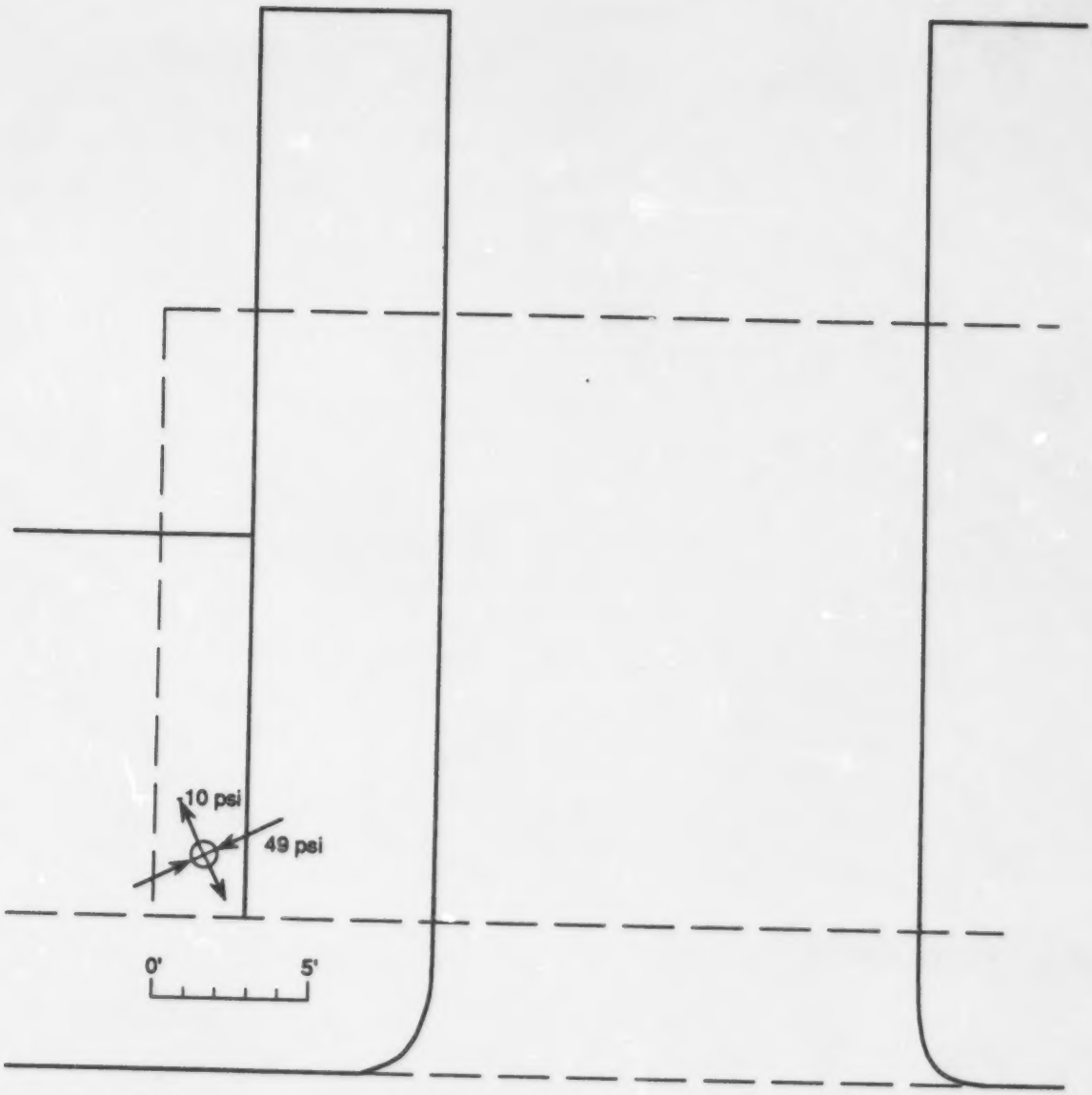
BH1
Test 1
Depth: 16'11"
Elev: 889'6"
E = 7.01×10^6 psi

FREDERICK HOUSE LAKE DAM



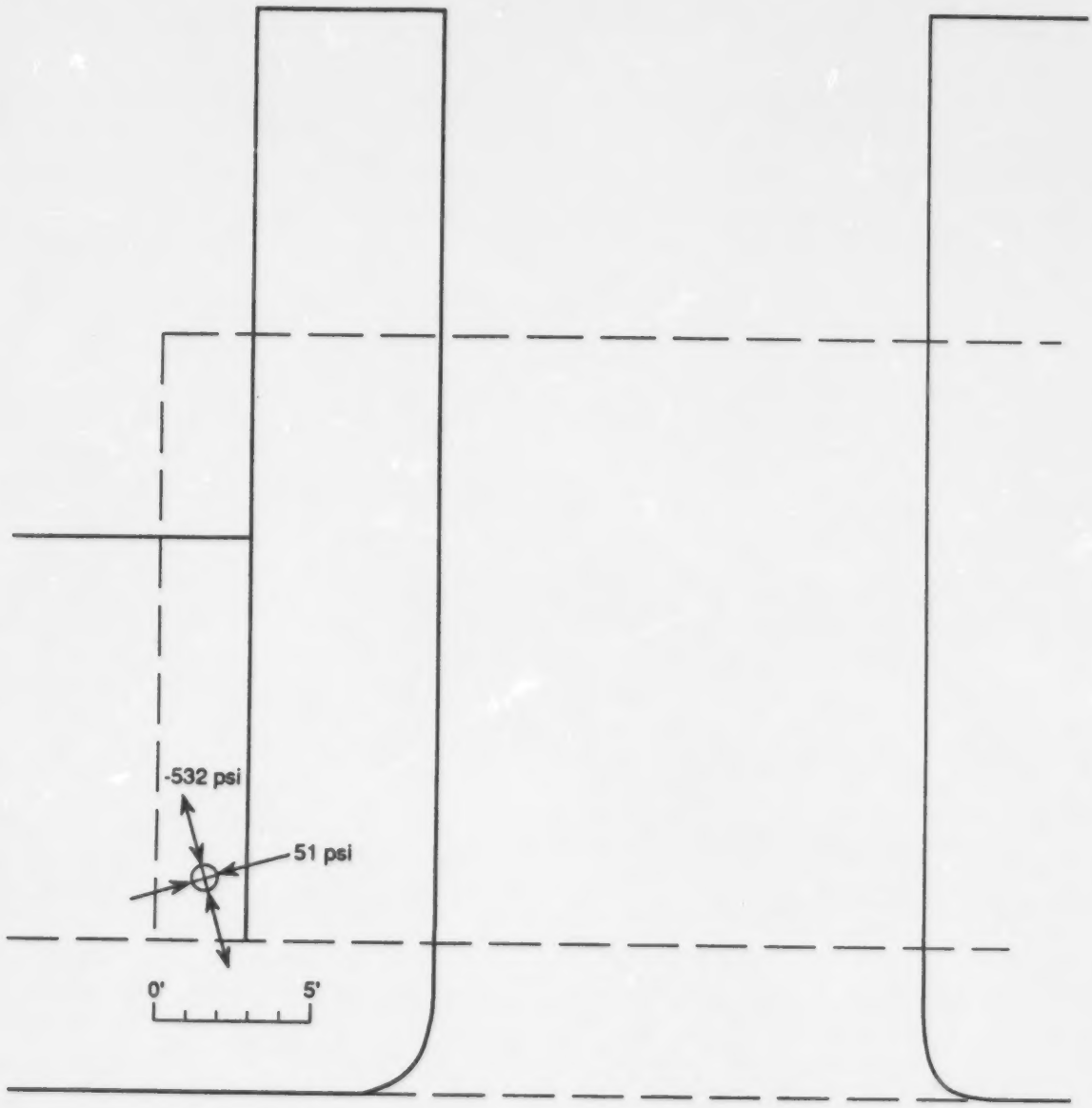
BH1
Test 2
Depth: 18'7"
Elev: 887'10"
E = 6.67 x 10^6 psi

FREDERICK HOUSE LAKE DAM



BH1
Test 3
Depth: 23'1"
Elev: 883'4"
E = 4.73×10^6 psi

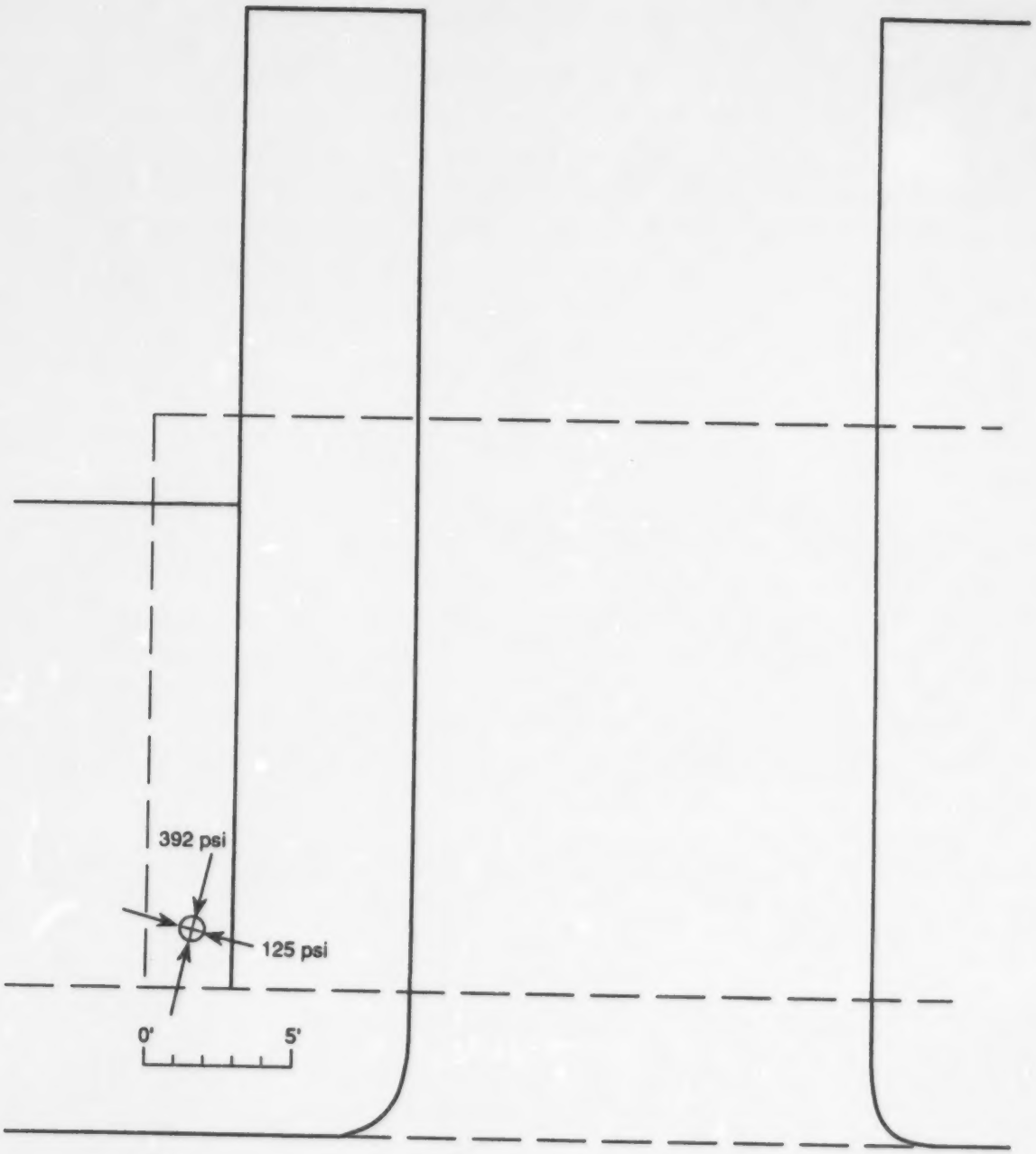
FREDERICK HOUSE LAKE DAM



BH1
Test 4
Depth: 24'1"
Elev: 882'4"
E = 4.73×10^6 psi

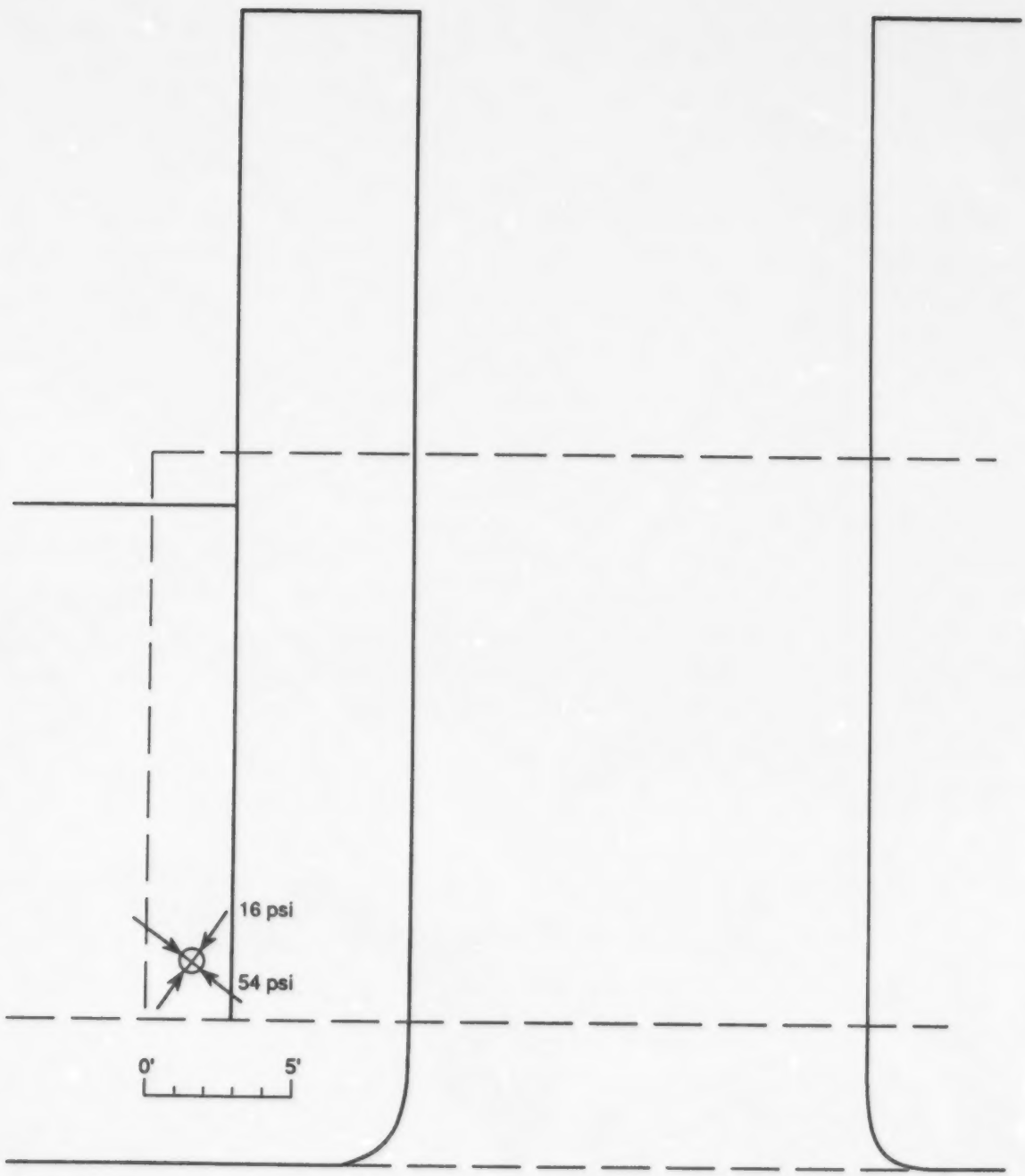
FREDERICK HOUSE LAKE DAM

182204-L11-RD
90-213



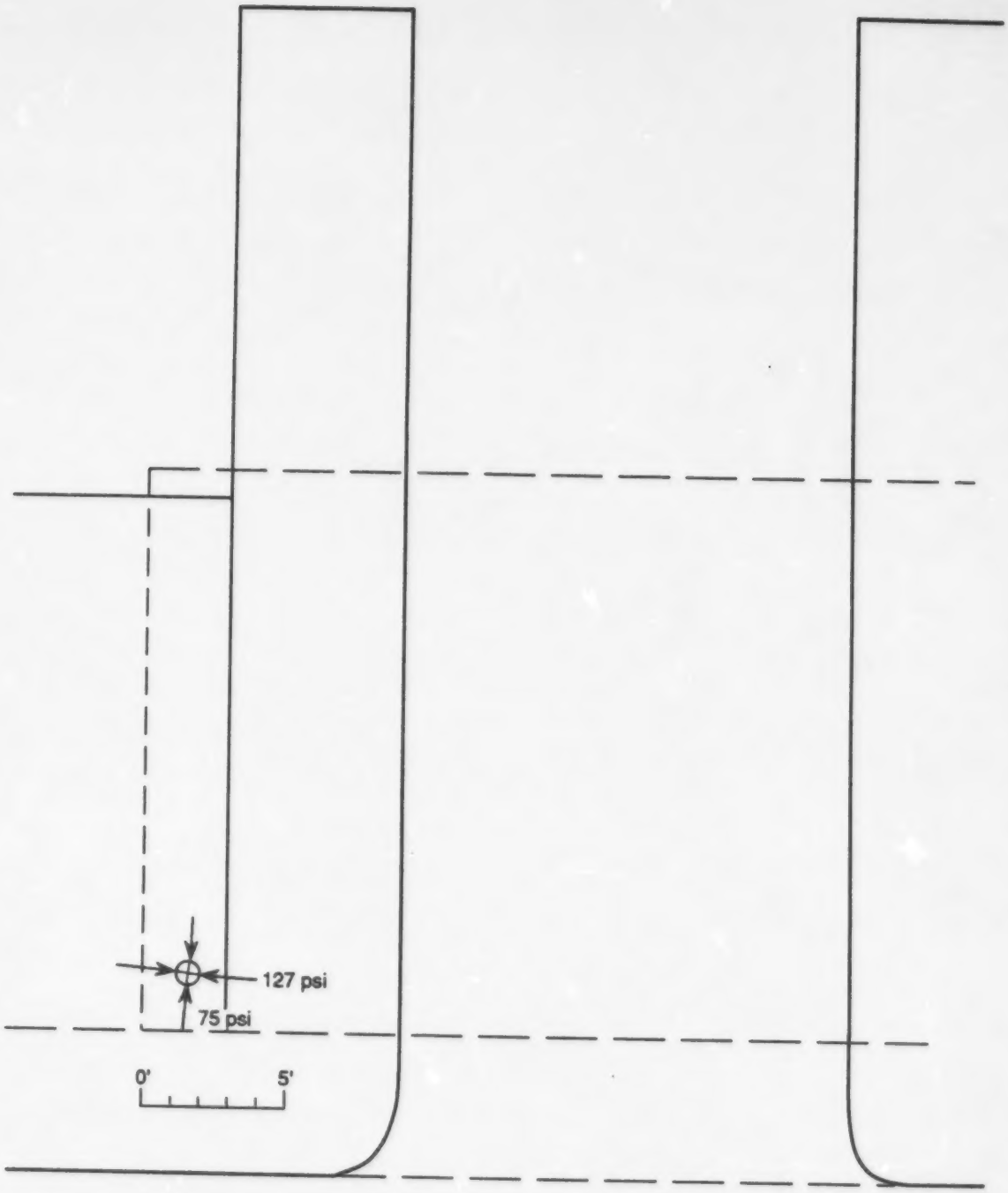
BH1
Test 5
Depth: 29'2"
Elev: 877'3"
E = 2.93×10^6 psi

FREDERICK HOUSE LAKE DAM



BH1
Test 6
Depth: 31'7"
Elev: 874'10"
E = 2.73×10^6 psi

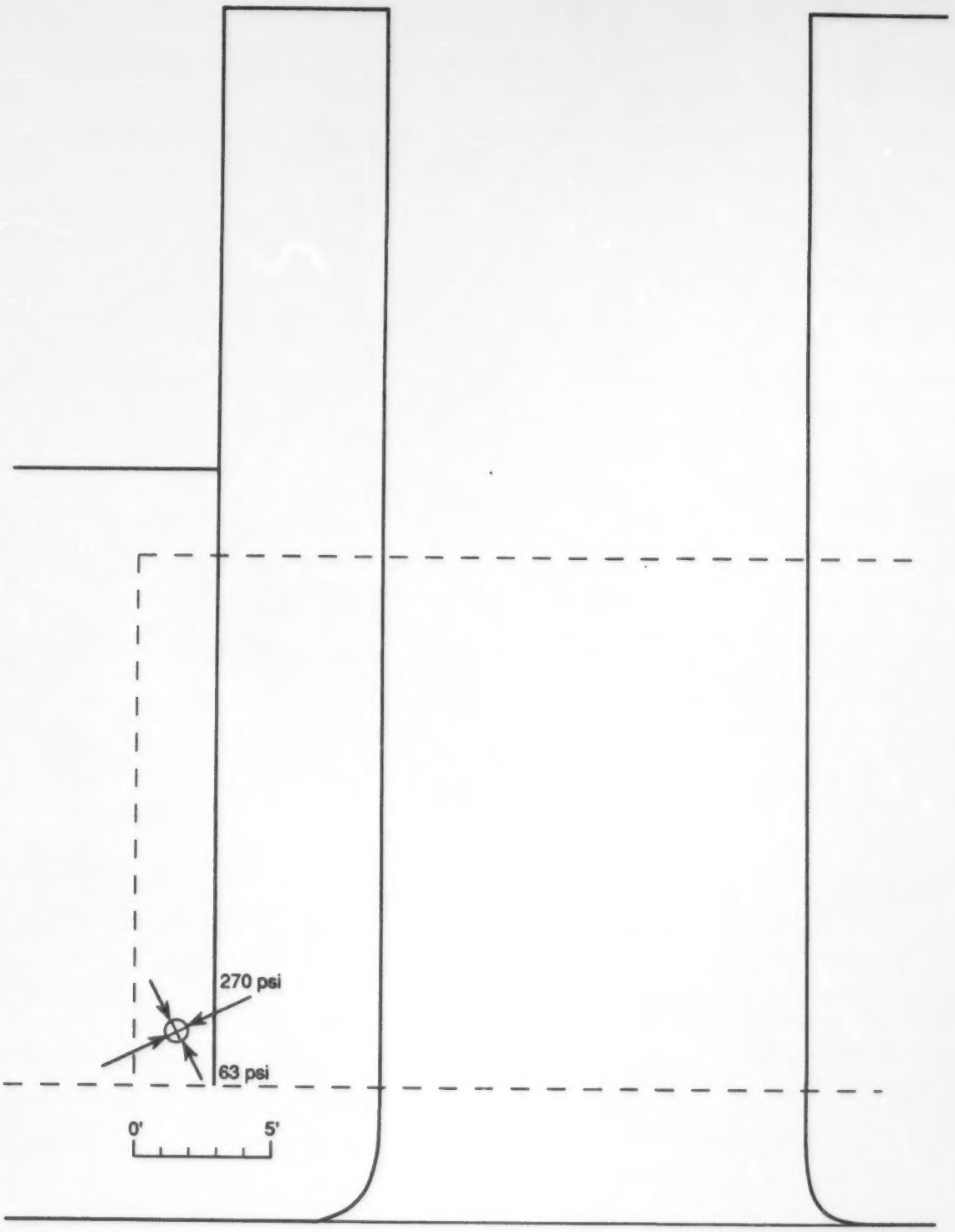
FREDERICK HOUSE LAKE DAM



BH1
Test 7
Depth: 32'10"
Elev: 873'7"
E = 2.73×10^6 psi

FREDERICK HOUSE LAKE DAM

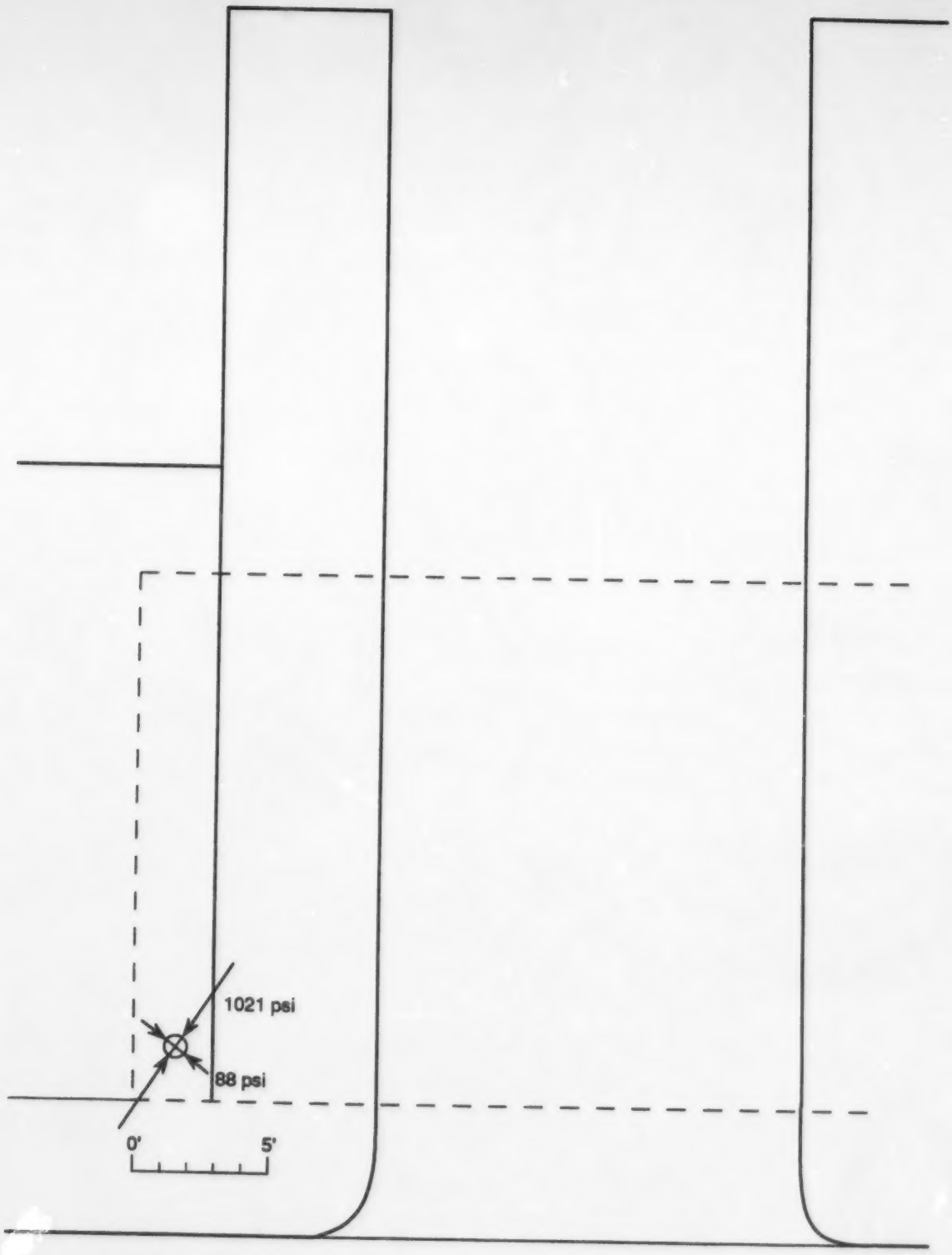
182207-L11-RD
90-213



BH1
Test 8
Depth: 38'11"
Elev: 867'6"
E = 3.65×10^6 psi

FREDERICK HOUSE LAKE DAM

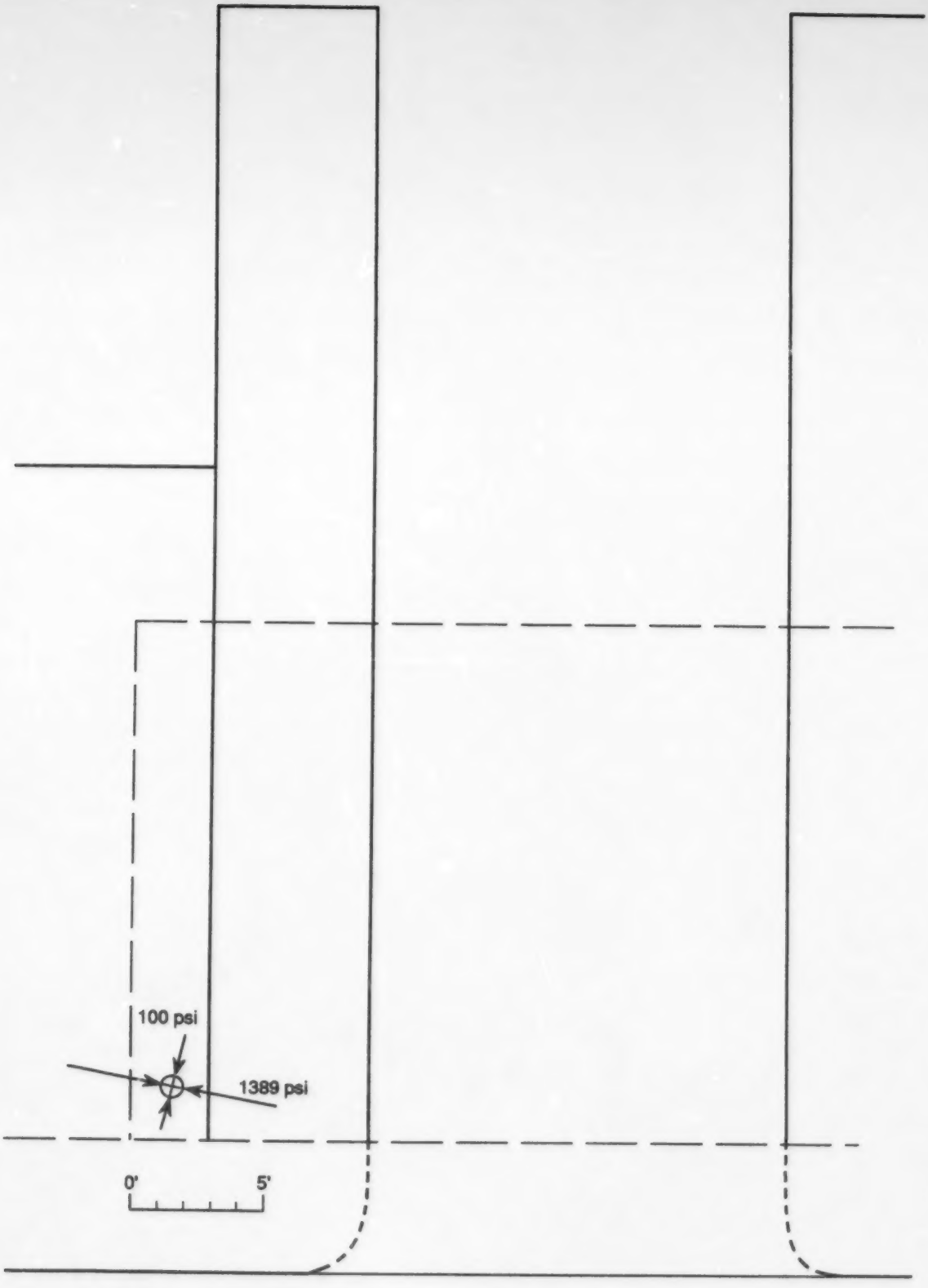
182208-L11-RD
90-213



BH1
Test 9
Depth: 40'1""
Elev: 866'4"
E = 3.65 x 10^6 psi

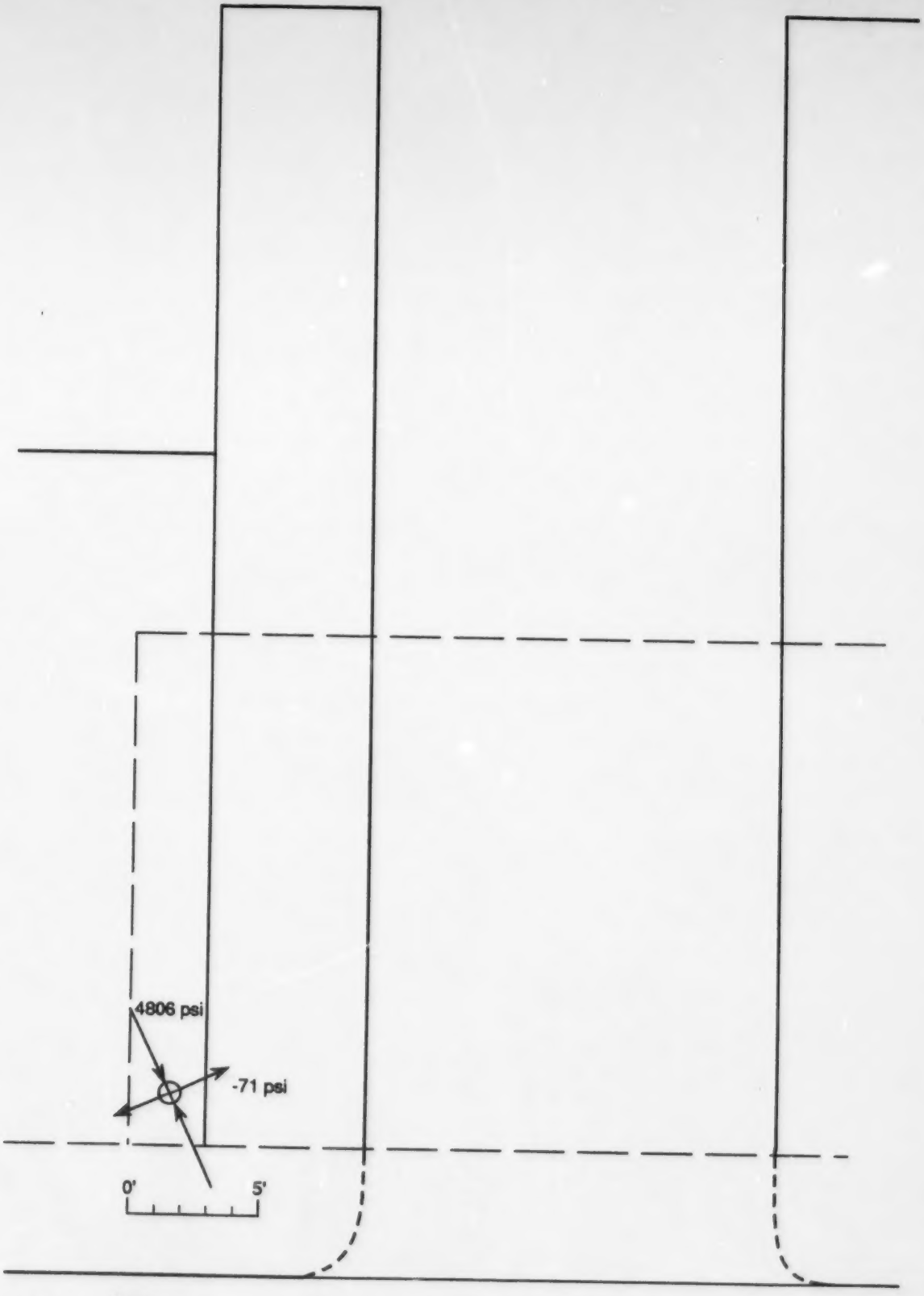
FREDERICK HOUSE LAKE DAM

182209-L11-RD

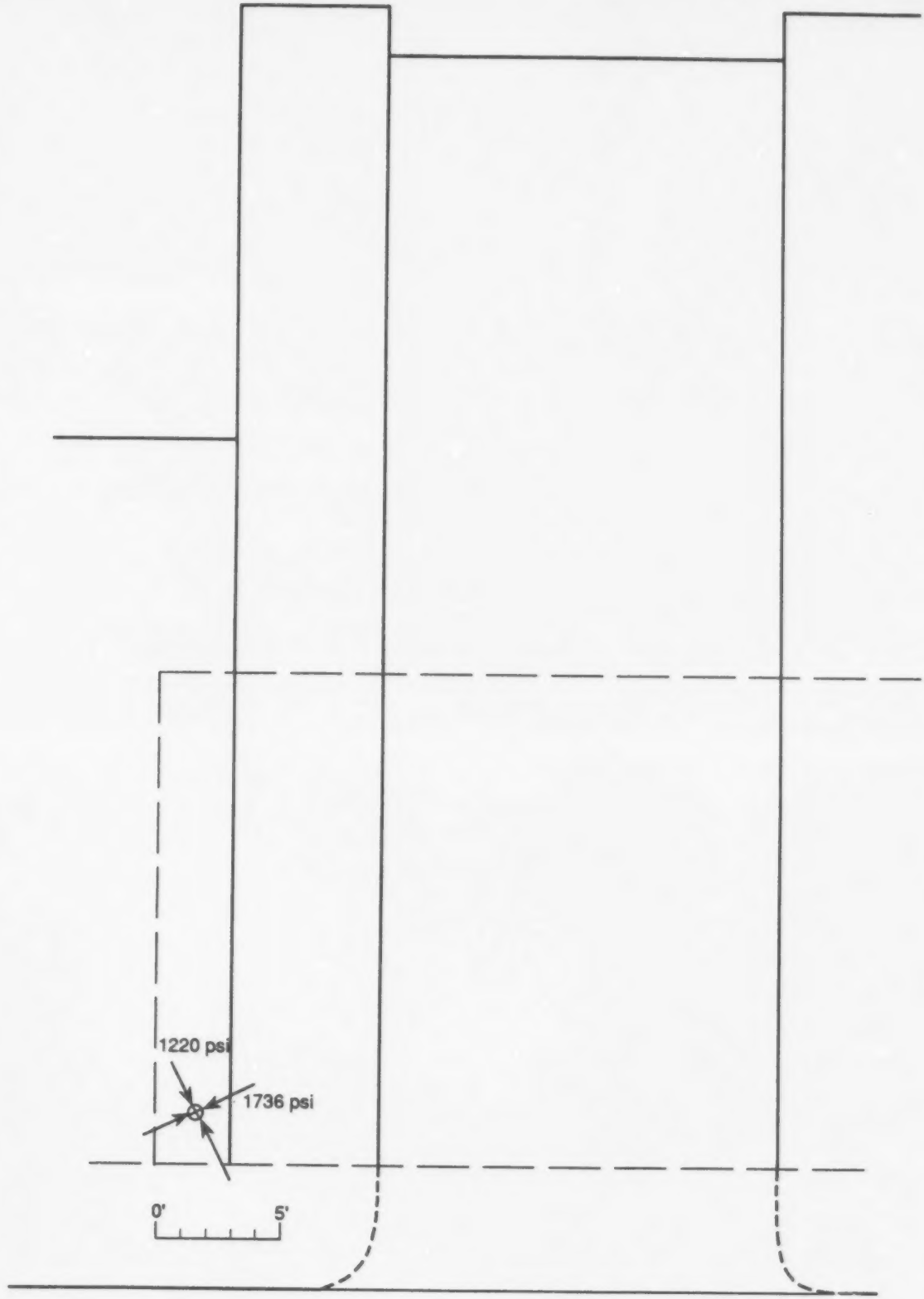


BH1
Test 10
Depth: 42'6"
Elev: 863'11"
E = 3.33 x 10^6 psi

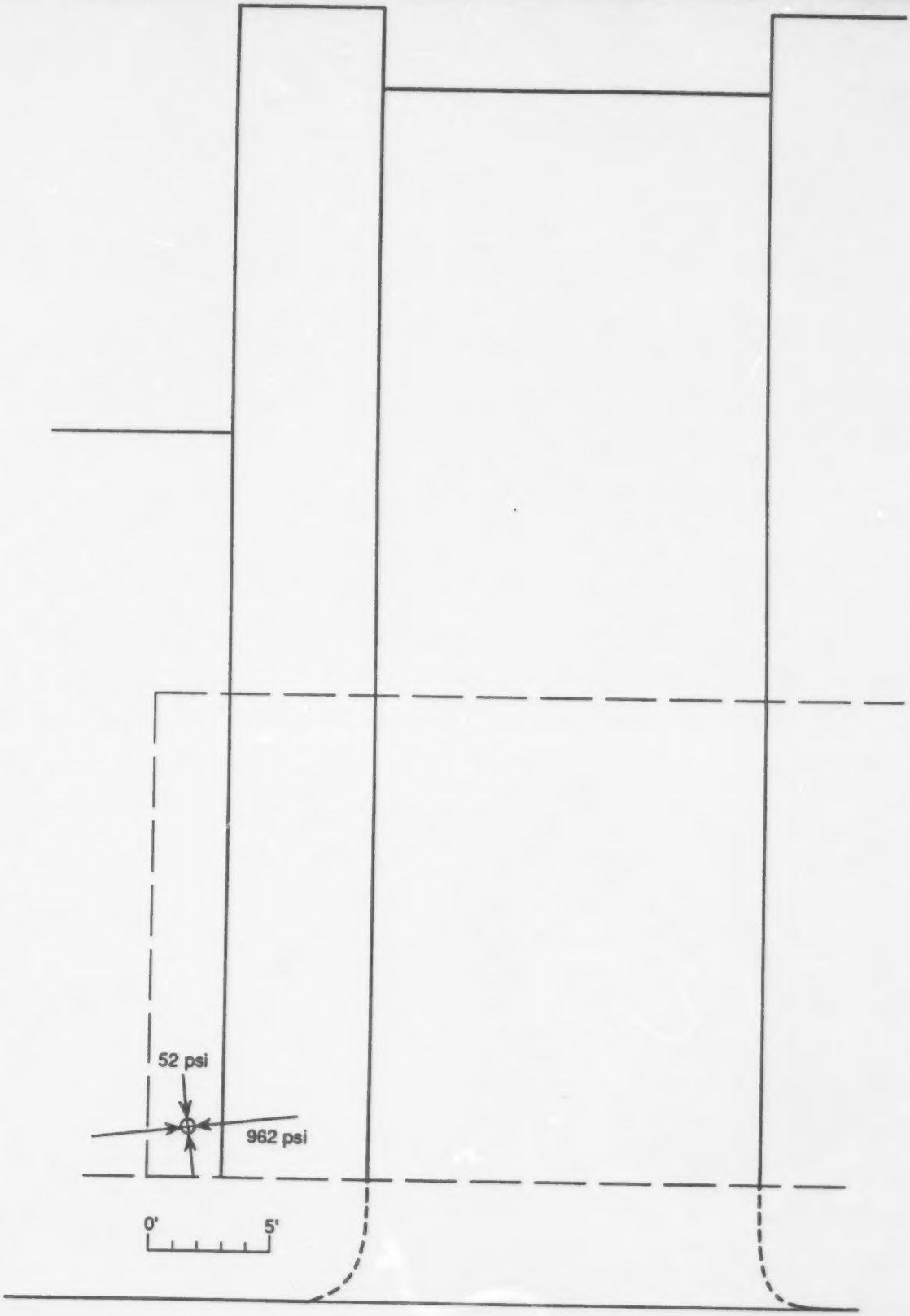
FREDERICK HOUSE LAKE DAM



BH1
Test 11
Depth: 43'8"
Elev: 862'9"
E = 4.41 x 10^6 psi

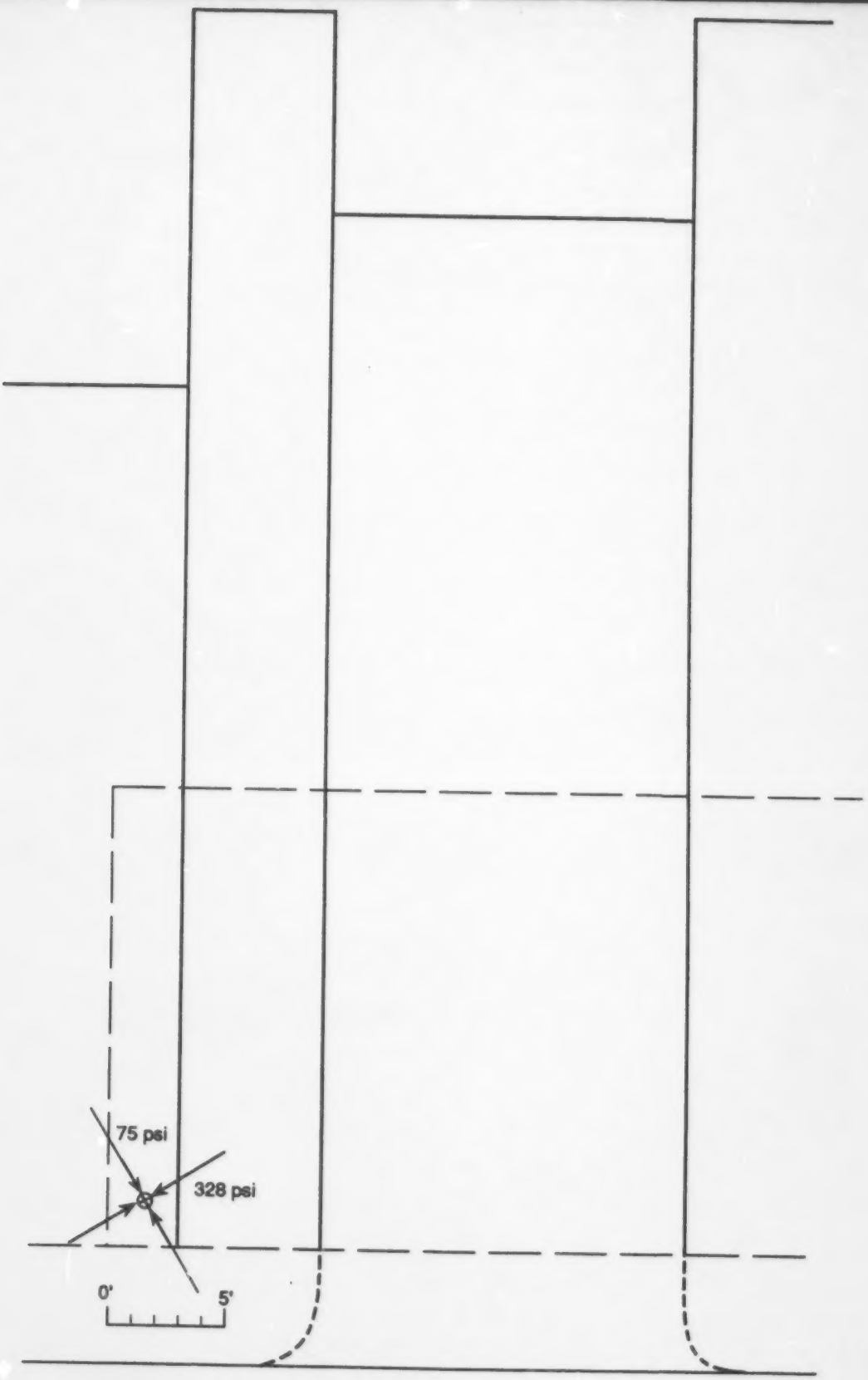


BH1
Test 12
Depth: 47'3"
Elev: 859'2"
E = 4.89 x 10^6 psi

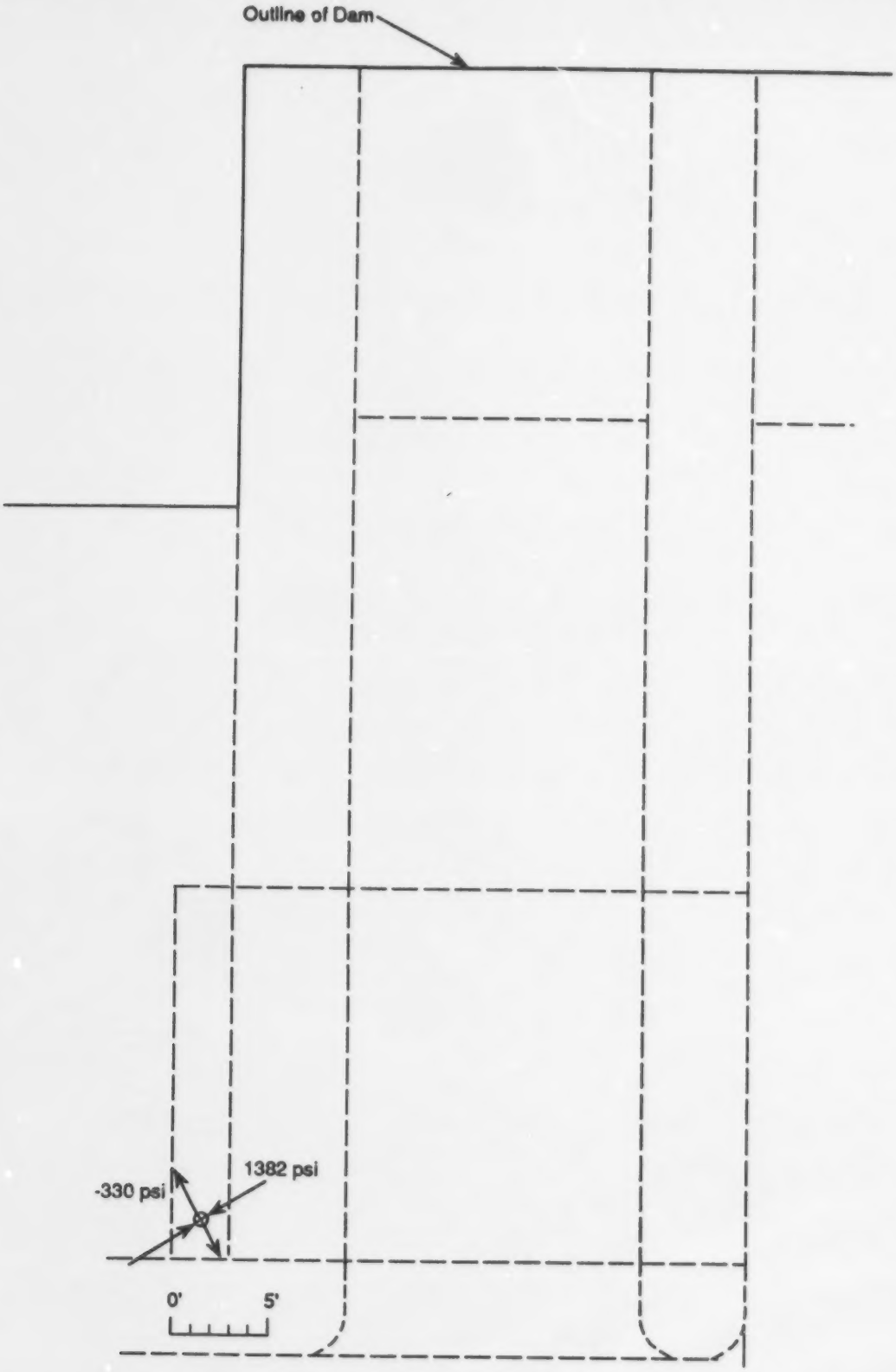


BH1
Test 13
Depth: 48'9"
Elev: 857'8"
E = 5.79×10^6 psi

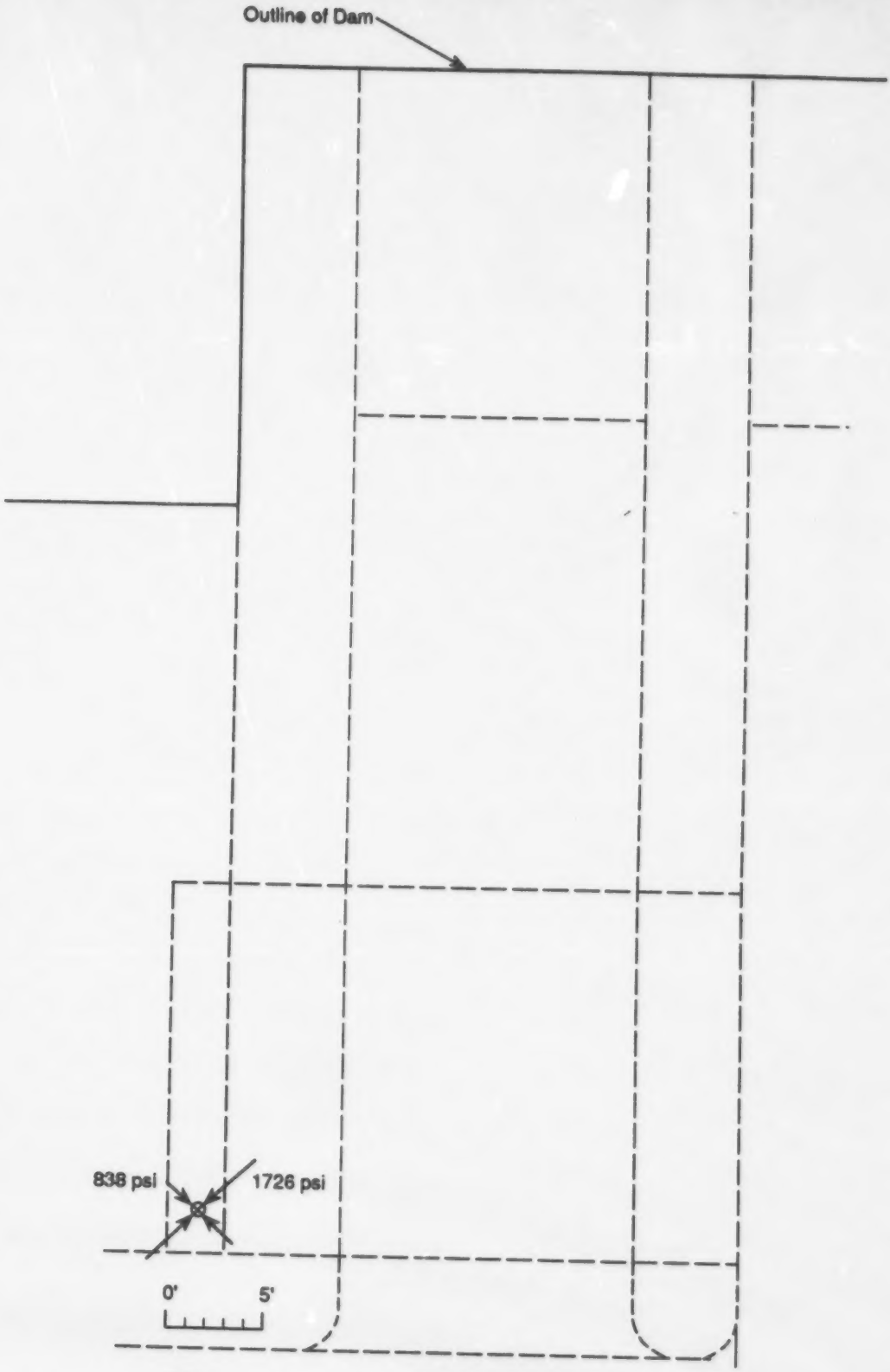
BH1
Test 14
Depth: 55'0"
Elev: 851'5"
E = 4.46 x 10^6 psi



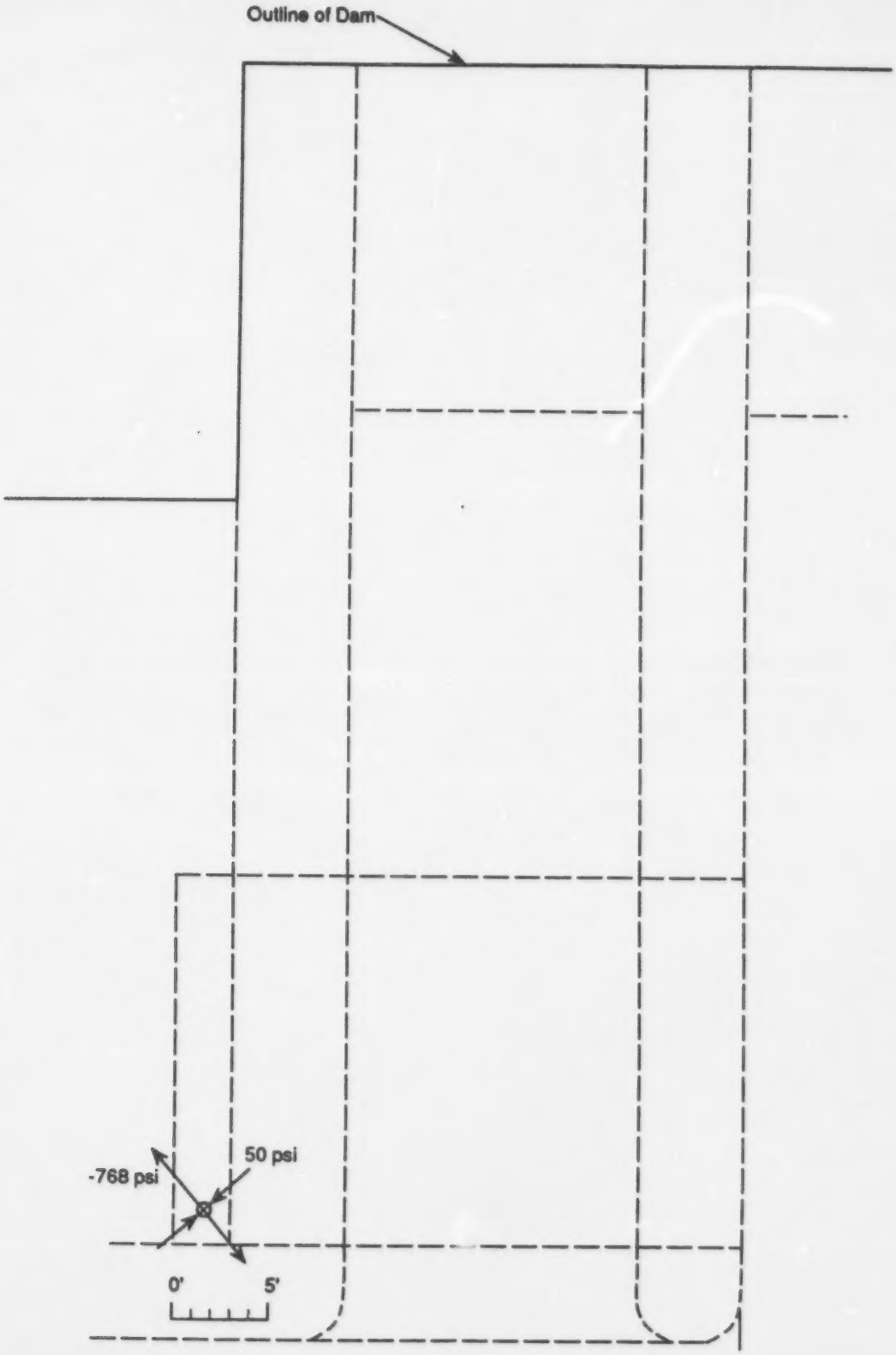
BH1
Test 15
Depth: 56'6"
Elev: 849'11"
E = 4.48 x 10^6 psi



BH1
Test 16
Depth: 66'1"
Elev: 840'4"
E = 12.33×10^6 psi

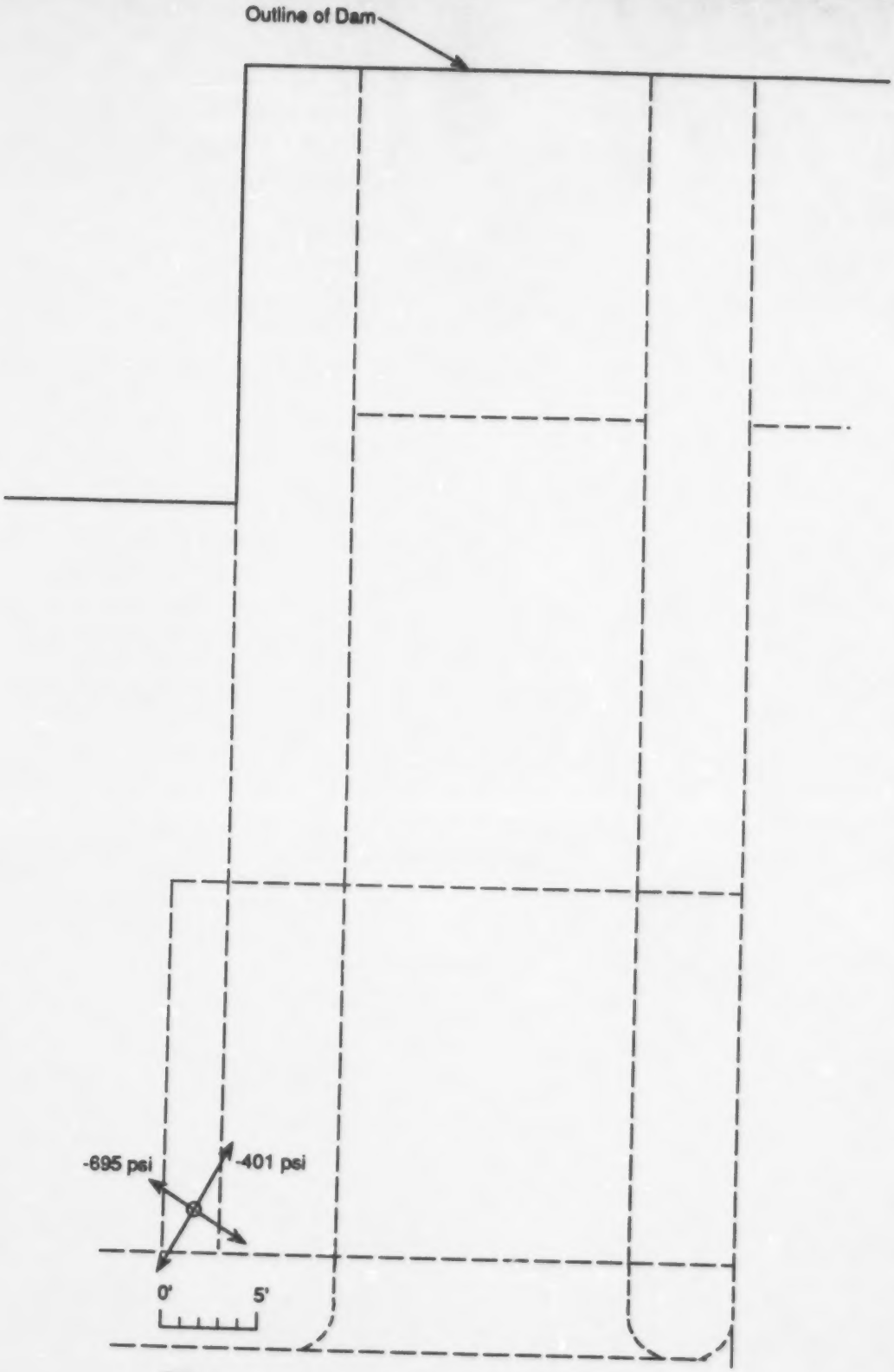


BH1
Test 17
Depth: 67'10"
Elev: 838'7"
E = 12.53×10^6 psi



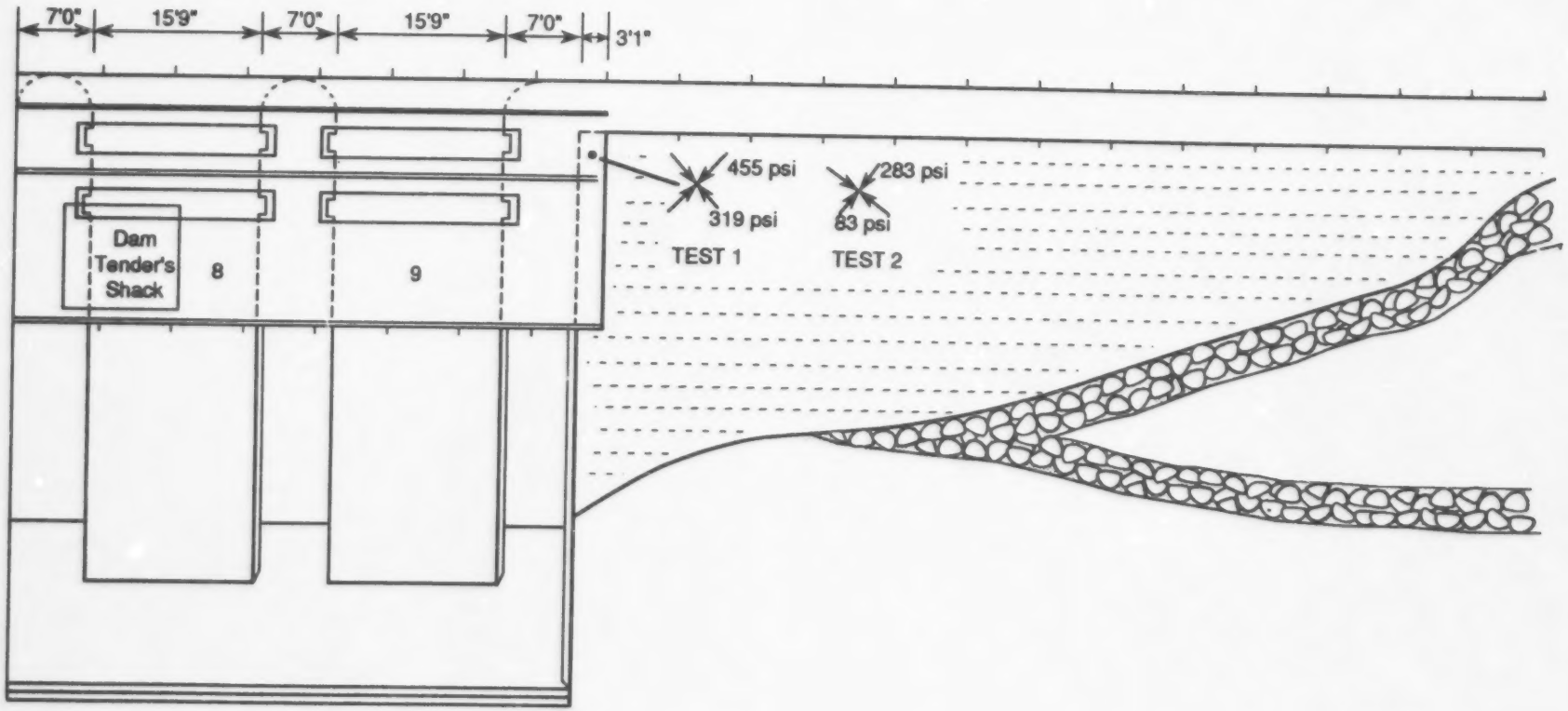
BH1
Test 18
Depth: 85'0"
Elev: 821'5"
E = 11.72×10^6 psi

FREDERICK HOUSE LAKE DAM


Outline of Dam
-695 psi
-401 psi
0'
5'
BH1
Test 19
Depth: 56'6"
Elev: 818'1"
E = 10.98 x 10^6 psi

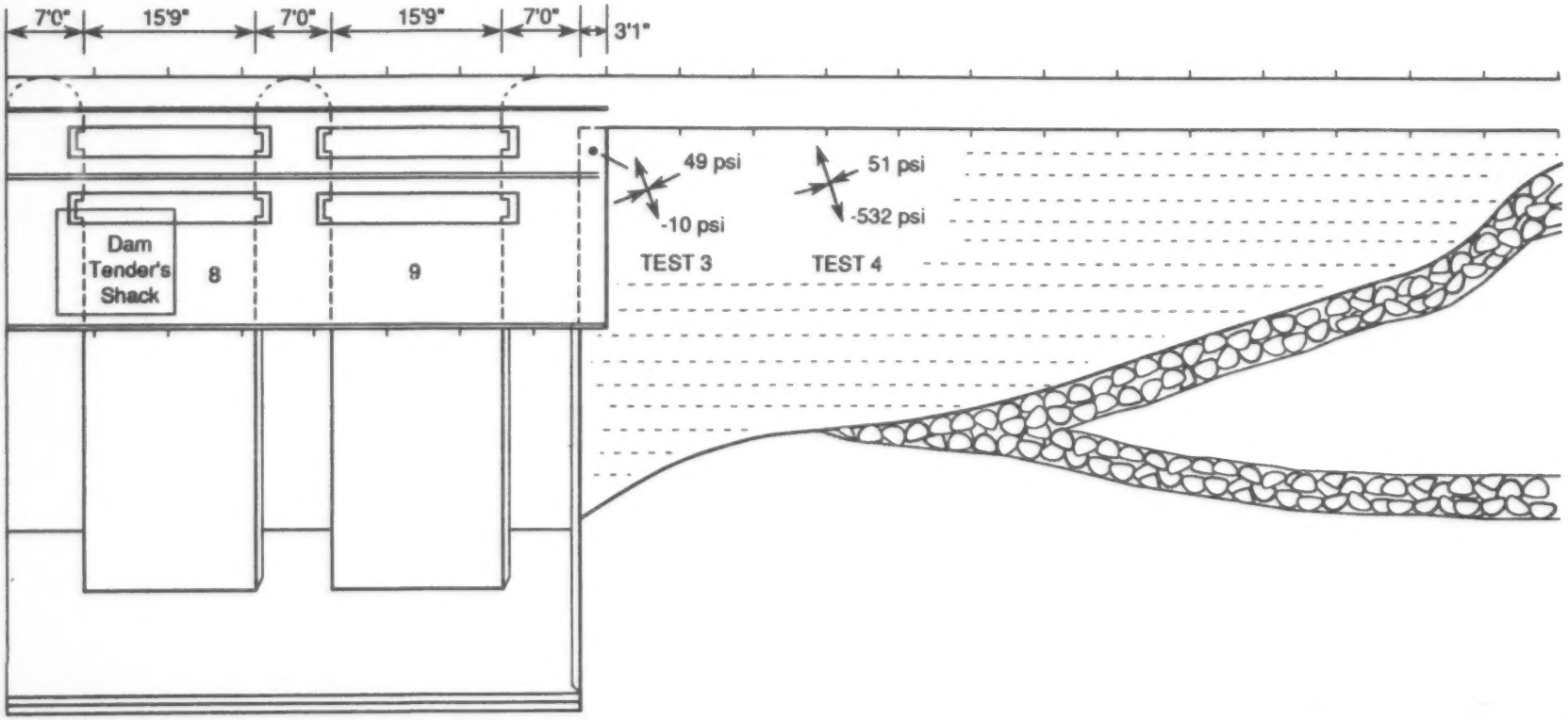
APPENDIX 5

TEST RESULTS - CROSS SECTION VIEW



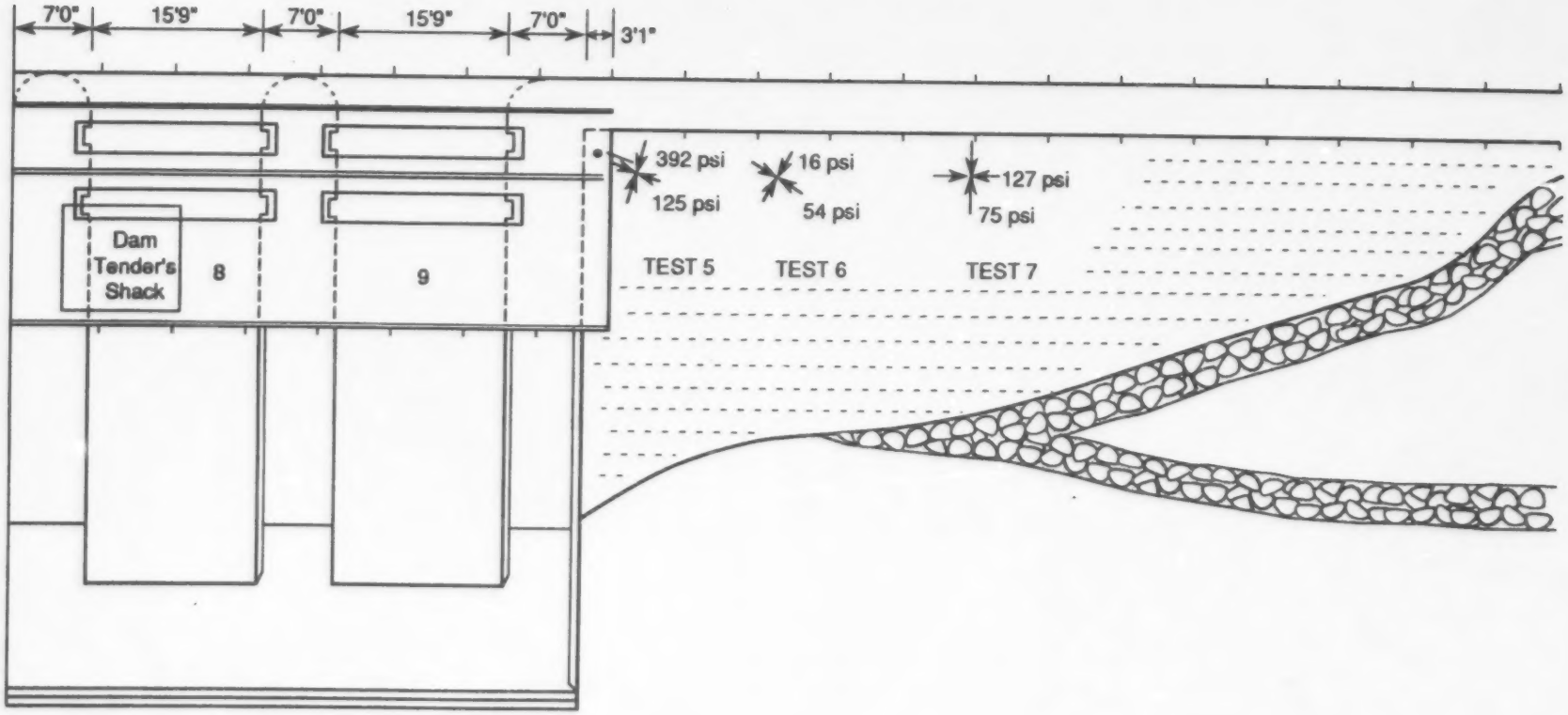
FREDERICK HOUSE LAKE DAM
OVERCORE TEST RESULTS
JULY 1989

90-213
18247-L11-RD

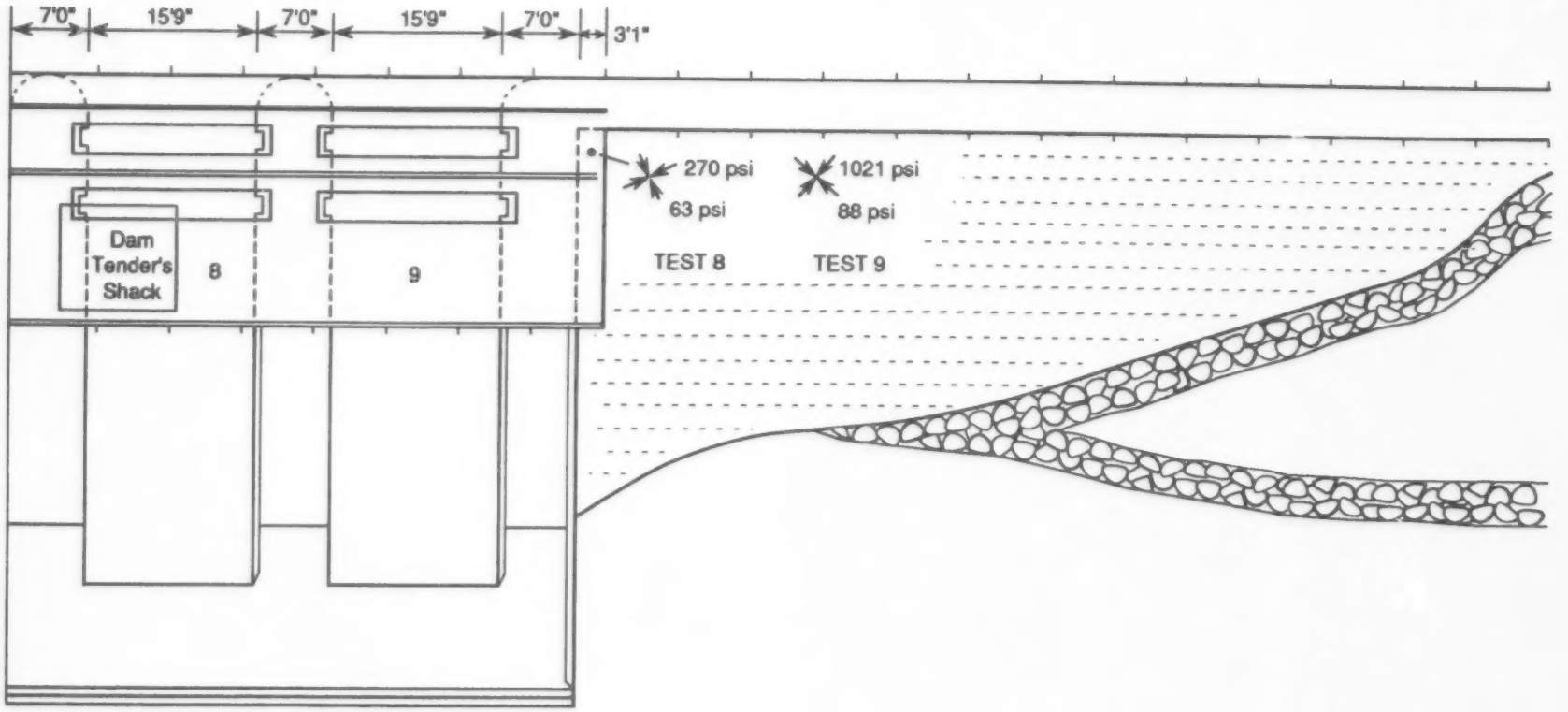


FREDERICK HOUSE LAKE DAM
OVERCORE TEST RESULTS
JULY 1989

18224B-L11-RD
90-213

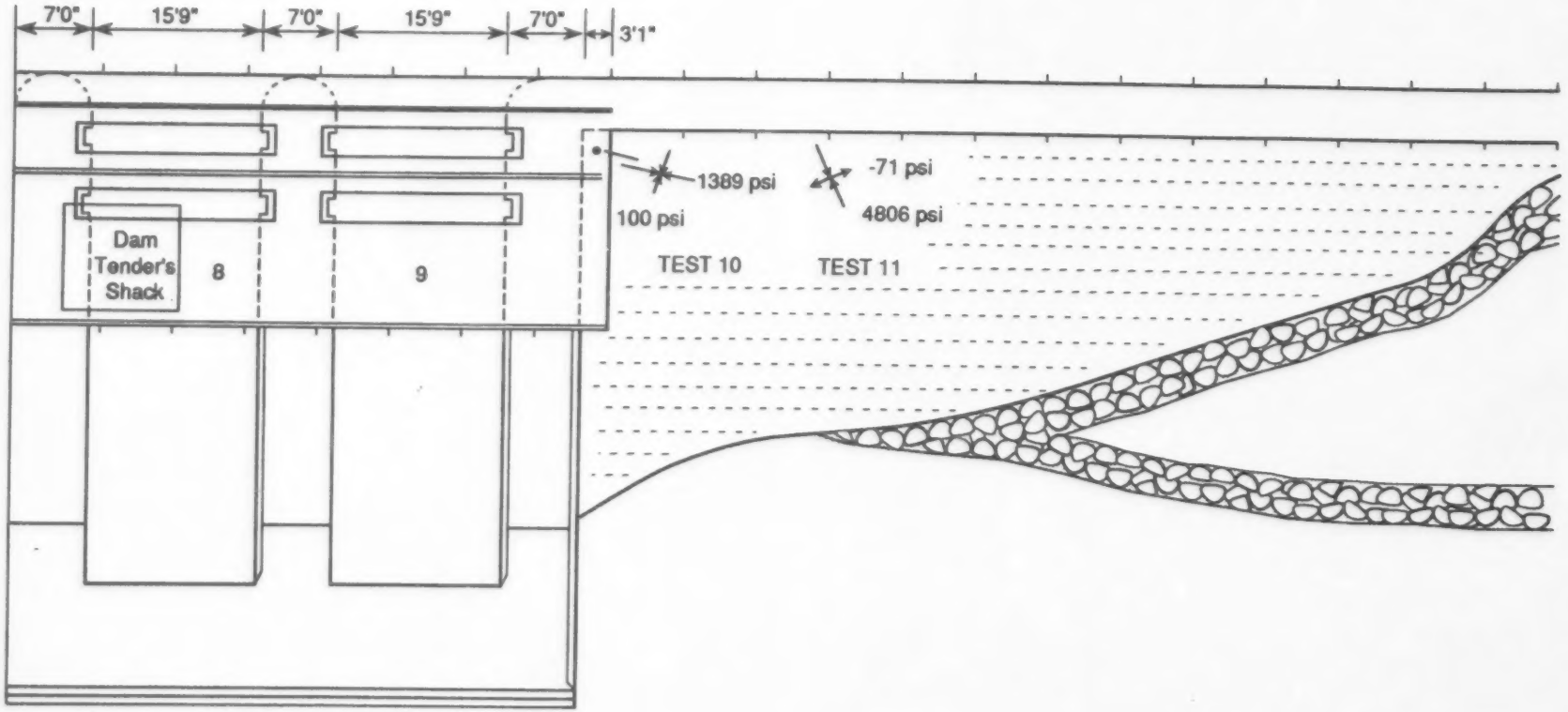


FREDERICK HOUSE LAKE DAM
OVERCORE TEST RESULTS
JULY 1989



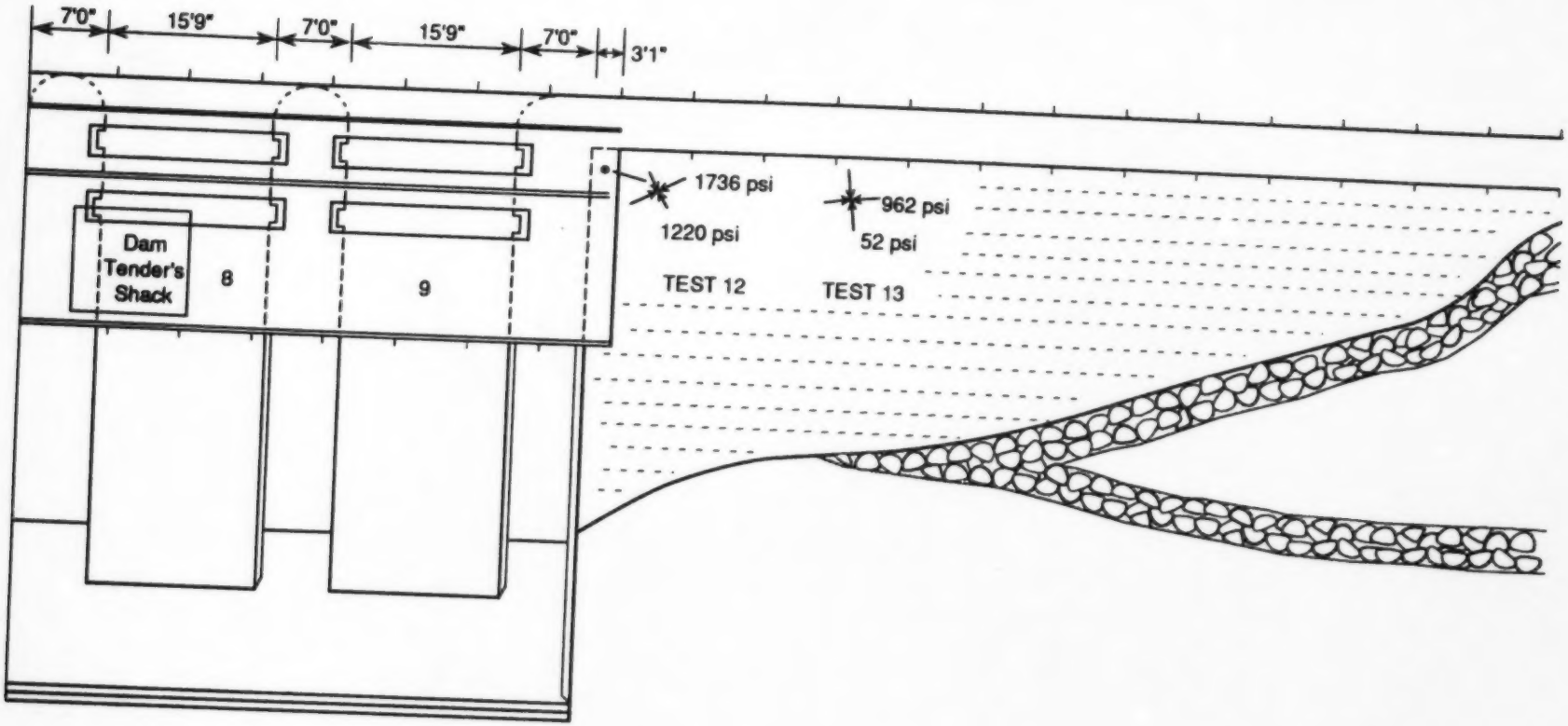
FREDERICK HOUSE LAKE DAM
OVERCORE TEST RESULTS
JULY 1989

182250-L11-RD
90-213

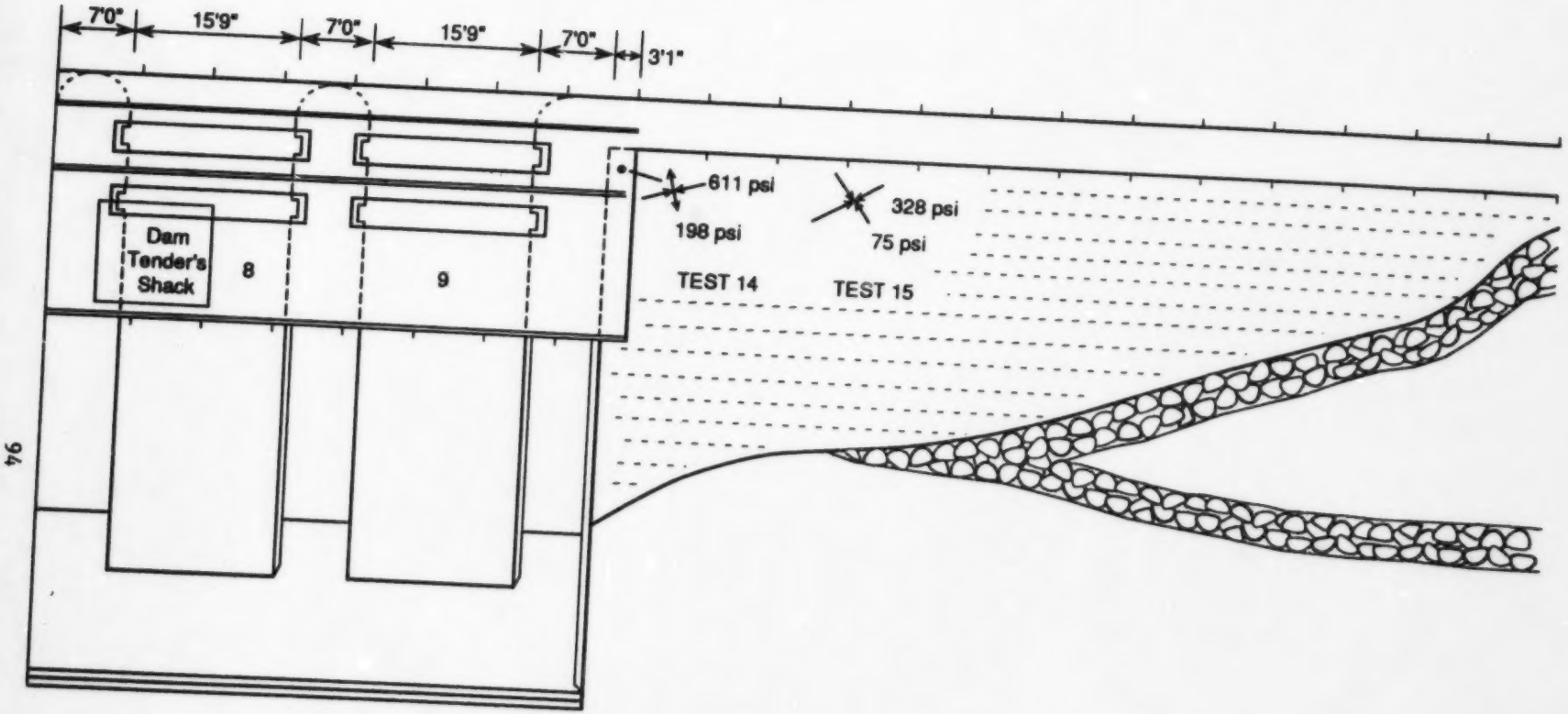


FREDERICK HOUSE LAKE DAM
OVERCORE TEST RESULTS
JULY 1989

90-21 8225-L11-RD



FREDERICK HOUSE LAKE DAM
OVERCORE TEST RESULTS
JULY 1989



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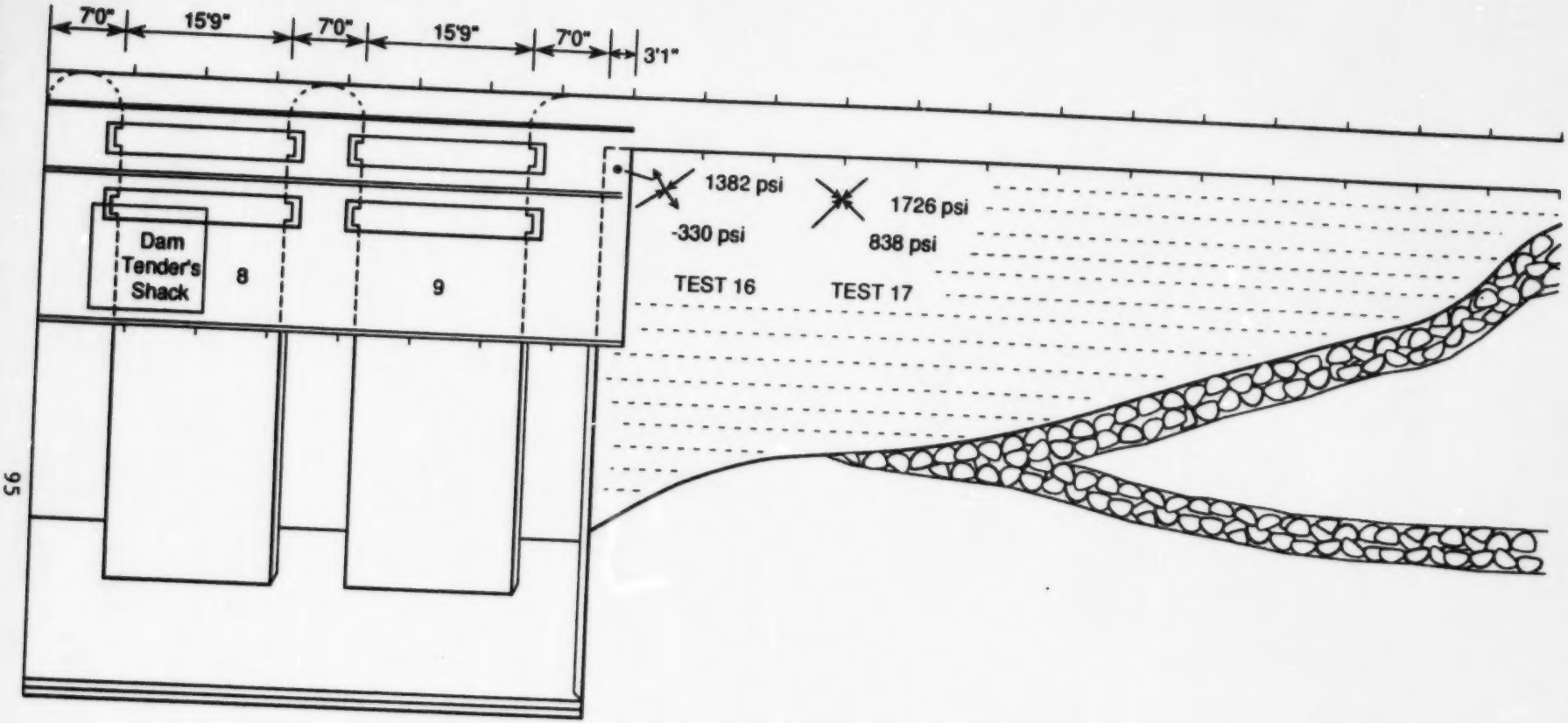
JULY 1989



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OVERCORE TEST RESULTS
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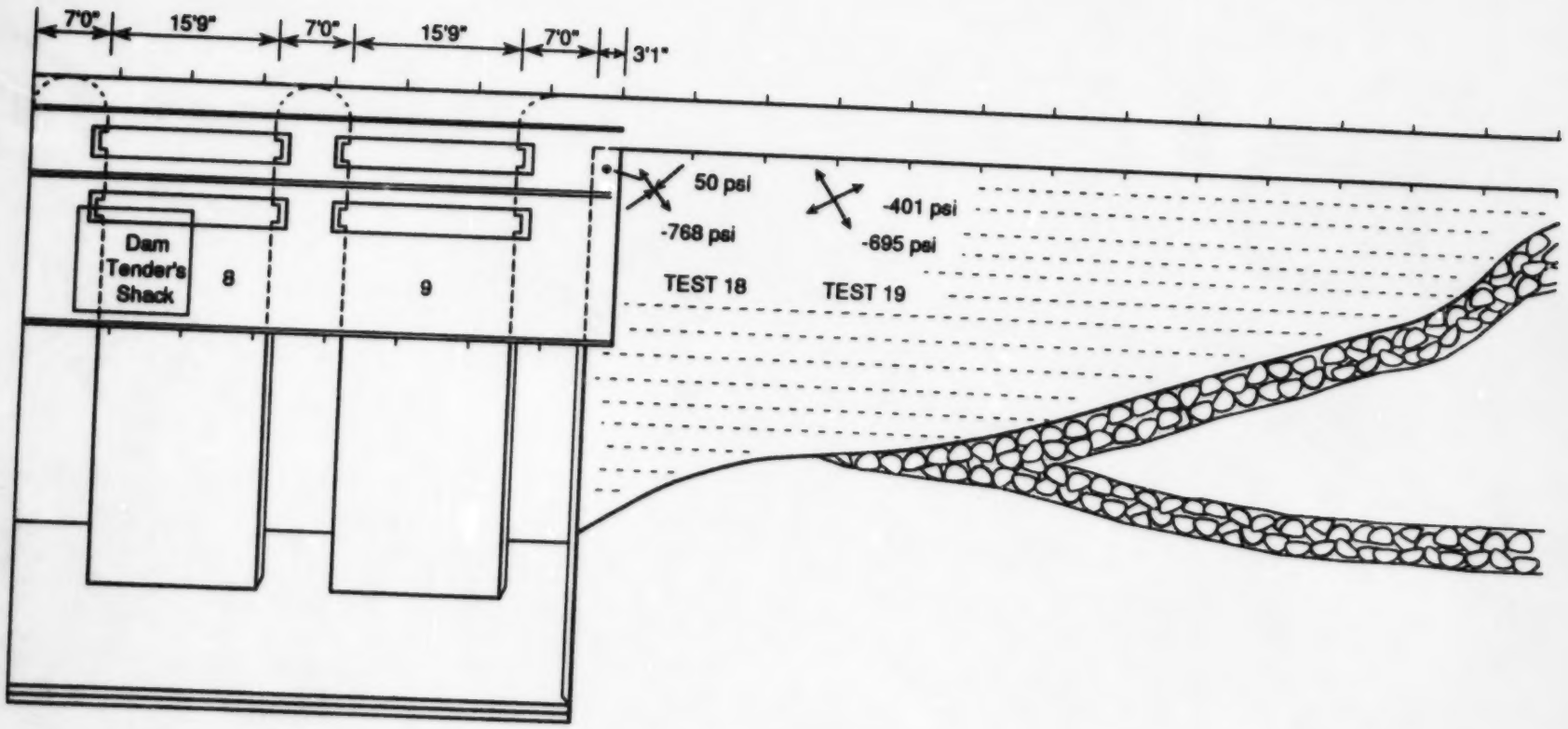

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OVERCORE TEST RESULTS
JULY 1989

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